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ALTERNATIVE ONSITE WASTEWATER TREATMENT AND DISPOSAL SYSTEMS ON SEVERELY LIMITED SITES

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16. ABSTRACT

Several research and evaluation studies were performed on alternative onsite systems on sites with severe limitations for conventional systems. The major limitations included slow permeability, a seasonally high water table, and limiting soil horizons. Several typical site evaluation techniques were also evaluated, and groundwater contamination was investigated.

The studies evaluated systems that rely on the soil for treatment and disposal (including Low Pressure Pipe, alternating soil absorption fields, shallow conventional trenches, gravelless trenches, and mounds) and systems that discharge to surface waters or to the atmosphere (including intermittent sand filters, upflow gravel filters, subsurface gravel beds, and evapotranspiration beds). Studies were performed on full-scale operating systems, scaled-down field systems, or laboratory columns.

A shallow trench system with sufficient volume to accommodate either a low loading rate or storage capacity for low absorption periods, had the highest potential for success for subsurface discharge. Alternating soil absorption systems (ASAS) could not be evaluated in these studies, but the use of shallow ASASs could potentially be highly successful. Intermittent sand filters preceded by a septic tank and followed by disinfection, as required, had the highest potential for success for surface discharge.

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FOREWORD

The U.S. Environmental Protection Agency is charged by Congress with protecting the Nation's land, air, and water systems. Under a mandate of national environmental laws, the agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. The Clean Water Act, the Safe Drinking Water Act, and the Toxic Substances Control Act are three of the major congressional laws that provide the framework for restoring and maintaining the integrity of our Nation's water, for preserving and enhancing the water we drink, and for protecting the environment from toxic substances. These laws direct the EPA to perform research to define our environmental problems, measure the impacts, and search for solutions.

The Water Engineering Research Laboratory is that component of EPA's Research and Development program concerned with preventing, treating, and managing municipal and industrial wastewater discharges; establishing practices to control and remove contaminants from drinking water and to prevent its deterioration during storage and distribution; and assessing the nature and controllability of releases of toxic substances to the air, water, and land from manufacturing processes and subsequent product uses. This publication is one of the products of that research and provides a vital communication link between the researcher and the user community.

This report presents research and evaluation studies performed on alternative onsite wastewater treatment and disposal systems in areas with severe limitations for conventional systems. This work will assist homeowners and regulatory agencies in evaluating available alternative systems for use in areas with similar soil and site conditions to those in this study.

ABSTRACT

Research and evaluation studies were performed on alternative onsite wastewater treatment and disposal systems in soils and on sites with severe limitations for the use of conventional systems. The studies included systems that rely on the soil for treatment and disposal (including low-pressure pipe systems, alternating soil absorption field systems, shallow-installed conventional trenches, gravelless trenches, and mound systems) and systems that do not rely on soils for disposal (including intermittent sand filters, upflow gravel filters, subsurface gravel beds, and evapotranspiration systems). These studies were performed on full-scale operating systems, scaled-down field systems, or laboratory columns that simulated the systems.

The soil properties, soil moisture regime, and shallow groundwater table in the area where the soil absorption systems were installed were characterized to gain a better understanding of the ability of the soils to treat and dispose of wastewater. Several typical site evaluation techniques were also evaluated. Because of the seasonally high groundwater table present in the soils in this study, groundwater contamination from wastewater was investigated.

Of the systems that were evaluated that rely on the soil for treatment and disposal of wastewater, the low-pressure pipe and the shallow-installed conventional systems absorbed a greater volume of effluent throughout the year than the gravelless or conventional trenches. However, all of the systems had periods of surface seepage during the winter months. Although a high groundwater table was often present in the soils in this study, there was no evidence of groundwater contamination from the effluent during the monitoring period.

The intermittent sand filters consistently produced the highest quality effluent with respect to BOD_5, TSS, and COD of all the surface discharge systems studied. Fecal coliform numbers were not reduced below 200 fecal coliforms per 100 ml in any of the effluent polishing units studied.

The soils retained a high percentage of water in the lower horizons throughout most of the year, even after the seasonally high groundwater table receded. This condition in which the soil was continuously near saturation or saturated further limited these soils for use as conventional soil absorption fields. The use of groundwater monitoring wells alone was not sufficient to identify this limiting condition.

Water movement in these soils occurred at a slow but acceptable rate for wastewater absorption when saturated or nearly saturated conditions were present. However, flow was nearly negligible when the soil became unsaturated. Flow from the soil absorption systems occurred in a saturated to nearly saturated state from the soil absorption systems throughout the year.

This report was submitted in fulfillment of Cooperative Agreement No. CR-808168 by the Cincinnati Center for Small Community Wastewater Systems Studies under the sponsorship of the U.S. Environmental Protection Agency. This report covers a period from October, 1980 to September, 1985, and work was completed as of October, 1986.

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SECTION 1

INTRODUCTION

Approximately one-fourth of all residences in the United States must rely on onsite wastewater treatment and disposal for domestic sewage disposal. Until recently, the conventional septic tank/soil absorption system was used by most homeowners because this was the only recognized onsite option available. Research has demonstrated that these systems can provide a reliable and efficient means of wastewater treatment and disposal at relatively low cost if they are properly sited, constructed, and maintained. Unfortunately, the soil conditions in approximately 68% of the total land area in the United States are unsuitable for onsite systems that use the soil for final treatment and disposal of wastewater. Hazardous health and environmental conditions have resulted from the installation of septic tank/soil absorption systems in these areas.

For the most part, homeowners struggled alone with the problems associated with onsite systems until the mid-1960's, when the great public awareness of the need to protect our environment elevated domestic sewage disposal problems to a community-wide concern.

In 1957, the Public Health Service published the Manual of Septic Tank Practice, which served as the state-of-the-art technical manual for many years. However, even at the time of the manual's revision in 1967, relatively little was known about how septic tank/soil absorption systems function, and even less information was available concerning alternative systems that might be used under unsuitable soil conditions. The U.S. Environmental Protection Agency (EPA) became involved with this area and initiated several projects (in particular, the Small Scale Waste Management Project at the University of Wisconsin) in the early 1970's to address these problems. Since then, many other studies have been undertaken and alternative systems have been developed.

Many of the newly developed successful alternative systems were custom-designed for the soil and site conditions at specific locations. Their use in other soil and site conditions is only slowly being evaluated. Also, many alternative systems have been demonstrated in local areas only, and officials elsewhere are reluctant to approve their use because the alternative systems are unproved and untested under the conditions that pertain to their areas. Clearly, more research is needed to establish the effectiveness of these alternative systems over a wide range of soil conditions.

The Cincinnati Center for Small Community Wastewater Systems Studies was established in 1980 in conjunction with the University of Cincinnati and the U.S. Environmental Protection Agency to provide expertise relevant to onsite wastewater disposal options for areas with unfavorable soil and site conditions such as those typically found in southwestern Ohio. The goals of the Cincinnati Center were accomplished by various methods, including demonstration, evaluation, and research projects.

Selected alternative designs that had not previously been evaluated in southwestern Ohio were investigated by the Center. In some of the studies, existing systems that had failed were replaced with alternative systems. In other studies, alternative systems that had already been installed but had not yet been proved to be acceptable alternatives in southwestern Ohio were evaluated. Research was also performed in situ on several scaled-down alternative systems and in laboratory columns simulating alternative systems. The alternative systems studied include those that rely on the soil for the treatment and disposal of wastewater and systems that do not rely on the soil for disposal but discharge either to surface waters or to the atmosphere. This document reports the findings of the Cincinnati Center.

SECTION 2

CONCLUSIONS

1. Most of the soils in southwestern Ohio have severe limitations for conventional methods of onsite treatment and disposal. The limitations include a seasonally high water table, limiting soil horizons, and slow permeability. Of these, the seasonally high water table appeared to be the most severe limitation throughout a 3-year study because soil absorption was drastically affected during the late fall, winter, and spring months when the water table was high.

2. Water flow in the Avonburg and Rossmoyne soils in this study occurred mainly in the macropores and was relatively stagnant in the micropores. Thus, when the soil was unsaturated and the macropores were excluded from the flow regime, water movement occurred very slowly.

3. The soil moisture content and potential below approximately 1 m in the soils studied did not reflect the low groundwater table during the summer, and the soil in the lower horizons appeared to remain nearly saturated throughout the year. Soil moisture monitoring equipment was required in addition to groundwater wells to characterize the soil moisture regime.

4. Failure to absorb sufficient wastewater occurred for up to 6 months of the year for conventional soil absorption trenches installed in an Avonburg soil.

5. Perimeter drains around soil absorption trenches installed in an Avonburg soil did not significantly increase the absorption rate of the soil.

6. A low-pressure pipe system installed in a Rossmoyne soil functioned satisfactorily throughout most of the year at a loading rate of 4 lpd/sq m, except during the winter months, when surface seepage periodically occurred.

7. Gravelless trenches that contained 25-cm corrugated plastic tubing performed similarily to conventional trenches in an Avonburg soil. However, neither trench design was capable of disposing of effluent during several months of the year, and therefore neither would be suitable for Avonburg and similar soils without modifications.

8. Shallow (35 cm deep) trenches covered with a mound of soil were more effective in disposing of wastewater than conventional trenches in an Avonburg soil. However, seasonal failure still occurred.

9. The installation of an alternating soil absorption field system resulted in the cessation of surface seepage during the 3-year study period at a site with a failing system. However, the trenches in the existing system had been installed with less than the recommended 90-cm separation distance between the trench bottom and bedrock. They were therefore inundated with groundwater during the late fall, winter, and spring regardless of whether effluent was applied to them, and rejuvenation of the existing system did not occur.

10. Groundwater contamination from several subsurface soil absorption systems was not a concern for the parameters of total phosphorus, fecal coliforms, ammonia, nitrite, nitrate, and total Kjeldahl nitrogen for the time period monitored.

11. To eliminate seasonal failure of subsurface absorption systems in soil and site conditions such as those present in southwestern Ohio, surface discharge of treated wastewater may be necessary.

12. Septic tanks or aerobic units alone were not capable of producing effluents that met local discharge standards.

13. Aerobic units followed first by upflow gravel filters and then by discharging gravel beds produced an effluent that attained Ohio discharge standards for BOD_5 and TSS. However, the level of treatment was not consistent and quality varied from day to day and from system to system.

14. Intermittent sand filters produced a high quality effluent with respect to biochemical oxygen demand, total suspended solids, and chemical oxygen demand when pretreatment was provided by either a septic tank or an aerobic unit. The additional expense of pretreating the wastewater with an aerobic unit was not justified.

15. Fecal coliform reduction from effluent polishing units (including intermittent sand filters, upflow gravel filters, and discharging gravel beds) was not sufficient to meet surface water standards for water contact recreational use of 200 fecal coliforms per 100 ml. Effluent from these units may require disinfection before surface discharge.

16. Three laboratory columns simulating mound soil absorption systems and dosed at 3-, 6-, and 12-hr intervals provided significant and equal treatment of septic tank effluent BOD_5, COD, suspended solids, fecal coliforms, and phosphorus. The column dosed at the 3-hr interval produced an effluent significantly lower in TKN, NH_3-N, NO_3-N, and total nitrogen concentration than did the other columns.

17. Three laboratory columns simulating recirculating sand filters with recirculation ratios of 3:1, 4:1, and 5:1 were effective in removing COD, TKN, and suspended solids from septic tank effluent. All recirculation ratios investigated gave the same degree of treatment.

18. Five pairs of laboratory columns simulating intermittent sand filters and dosed at hydraulic loading rates ranging from 20 to 160 lpd/sq m successfully treated septic tank effluent for COD, suspended solids, NH_3-N, and TKN. Phosphorus removal efficiencies tended to increase as the hydraulic loading rate on the columns decreased. No fecal coliforms were detected at hydraulic loadings less than 80 lpd/sq m. Total nitrogen levels were higher than the influent levels and tended to rise as the hydraulic loading rate decreased.

19. A survey of existing evapotranspiration systems in Hamilton County, Ohio, showed that 6 of the 11 systems identified were failing. These systems were not considered to be true evapotranspiration systems and they are not recommended in southwestern Ohio.

20. Soil morphological features, including mottles and iron and manganese nodules and cutans, could generally be used to predict the presence and duration of the high water table in the Avonburg and Rossmoyne soils in this study. Using only mottles with chromas of 2 or less to predict the depth to the high water table would have underestimated the actual height of rise because the groundwater table was observed in horizons that did not contain mottles with chromas of 2 or less.

21. Percolation tests, in situ falling-head permeability tests, and auger-hole hydraulic conductivity tests performed on Avonburg and Rossmoyne soils produced highly variable results that were not significantly different. All of the tests predicted a higher loading rate than actually occurred.

SECTION 3

RECOMMENDATIONS

1. Further evaluate the low-pressure pipe system for onsite wastewater treatment and disposal in the soil and site conditions present in southwestern Ohio. Use a loading rate of 4 lpd/sq m or less, and, if possible, use a wider spacing between trenches than typically recommended.

2. In slowly permeable soils with seasonally high water tables install subsurface soil absorption trenches in the upper soil horizons to utilize the more permeable topsoil and to provide the greatest possible separation distance between the high groundwater table and the infiltrative surface. Continue monitoring shallow systems to determine the depth of soil cover required to prevent freezing.

3. Determine whether using wider trench spacing in soils with high capillarities could increase the amount of effluent that can be applied to a soil.

4. Do not install evapotranspiration systems in areas such as southwestern Ohio where precipitation exceeds evapotranspiration for any time of the year.

5. Monitor the groundwater around soil absorption systems installed in soils with high groundwater tables for longer than 1 yr to determine whether untreated effluent contaminates the groundwater.

6. Evaluate the effectiveness of perimeter drains for increasing the wastewater absorption capacity of slowly permeable soils with seasonally high water tables. Monitor absorption rates and soil moisture contents as well as groundwater table depths.

7. Include soil morphological features such as 3-chroma mottles, and Fe and Mn nodules and cutans in site evaluations to obtain a more accurate prediction of the presence and duration of the high groundwater table.

8. Use the percolation test only in conjunction with a detailed soil morphological description to determine site suitability for soil absorption or for sizing a system. Develop hydraulic conductivity curves for the major soil types in an area to provide more complete information about a soil's ability to absorb wastewater. Continue research on sizing systems using only soil morphological descriptions.

SECTION 4

DESCRIPTION OF RESEARCH AREA

GENERAL SITE CHARACTERISTICS

Climate

The climate of southwestern Ohio is characteristically humid, temperate, and continental. The winters are moderately cold with an average temperature of 0.5^{0} C and frequent periods of extensive cloudiness. The average temperature during the rather humid summer is 23^{0} C. The freeze-free period lasts on the average 187 days from mid-April to the latter part of October.

The average total annual precipitation is 102 cm. A precipitation maximum occurs during winter and spring with a late summer and fall minimum. The average relative humidity in midafternoon is approximately 60%. Humidity is higher at night, and the average at dawn is about 80%. The area receives 70% of the sunshine possible in summer, and 40% in winter (Lerch et al., 1982; National Oceanic and Atmospheric Administration, 1984).

Geology

Most of southwestern Ohio lies within the Till Plains section of the Interior Low Plains physiographic province (Bier, 1967). The main local physiographic features are nearly level to gently rolling glacial uplands, steep hillsides along the major streams, extensive glacial river terraces and outwash plains, and flood plains. The bedrock underlying the area is shale and fossiliferous limestone of Middle and Late Ordovician age. The bedrock outcrops on steep valley walls and at waterfalls, and in other areas is overlain by glacial deposits that range in thickness to as much as 120 m (Lerch et al., 1975, 1980, 1982).

Soils

At least three continental glaciers--the Kansan, the Illinoian, and the Wisconsinan--encroached upon southwestern Ohio. Wisconsinan loess covers the older Illinoian and Kansan tills over most of the areas that were not covered by the Wisconsinan glacier. The soils that make up a large percentage of the total acreage of the area formed in these deposits of Wisconsinan loess and the underlying Illinoian till. These soils have strongly developed profiles, high clay contents, and many have a fragipan, which is a compact, brittle layer in the subsoil, which impedes root penetration and percolation of water. Permeability is slow in these soils because of the fragipan and the high clay contents.

These and other soil and site characteristics limit the use of these soils as soil absorption fields for the treatment and disposal of wastewater. In Hamilton and Clermont counties, two of the most southwestern counties in Ohio, 88 and 96% of the soils, respectively, are severely limited for use as soil absorption fields (Lerch et al., 1975, 1982). The percentage of soils having specific limiting factors is given in Table 1.

TABLE 1. THE PERCENTAGE OF SOILS IN HAMILTON AND CLERMONT COUNTIES, OHIO, HAVING SPECIFIC LIMITATIONS FOR ONSITE WASTEWATER DISPOSAL

Limiting factor	% of Acreage*	
	Hamilton Co.	Clermont Co.
Slow permeability	75	68
Wetness	34	40
Slope	30	22
Depth to bedrock	22	6
Flooding	9	6
Poor filter	5	--

* Multiple limitations exist on some soils.

The majority of soils in these two counties have slow permeabilities, which limits their usefulness as soil absorption fields. The fragipan and high clay contents are among the factors that make these soils slowly permeable. The second widespread limiting factor is wetness, which often results from a seasonal high water table. Many of these soils are inundated with groundwater during the late fall, winter, and spring, and dry out very slowly in the summer months. Steep slopes and a shallow depth to bedrock are also major limiting factors for onsite wastewater treatment and disposal.

SPECIFIC SOIL CHARACTERISTICS

A field station was established in the northern section of Clermont County, Ohio (Figure 1), to perform research on alternative methods of subsurface wastewater treatment and disposal. The 1.2-hectare Illinoian till plain site was north-facing and sloped toward a stream tributary. A contour map of the site is shown in Figure 2. The southern portion of the site was nearly level, with maximum slopes of 12% located in the northern section nearest the stream. The site was chosen because it contained two of the major soil types found in southwestern Ohio. These included Avonburg silt loam (Aeric Fragiaqualf, fine-silty, mixed, mesic) and Rossmoyne silt loam (Aquic Fragiudalf, fine-silty, mixed, mesic). These soils formed in 40 to 100 cm of Wisconsinan loess and in the underlying weathered glacial till of Illinoian age.

These two soil types were characterized in more detail at the research station. Soil pits were dug in each soil type, and detailed soil profile descriptions were made in each pit according to standard soil survey procedures (Soil Survey Staff, 1951). Samples were extracted from each pit to determine bulk density (Blake, 1965 - core method), particle-size analysis (Day, 1965 - hydrometer method), optimum moisture content (Bowles, 1978 - Standard Proctor Test), Atterberg limits (Bowles, 1978), and to obtain soil moisture retention curves (Bouma et al., 1974).

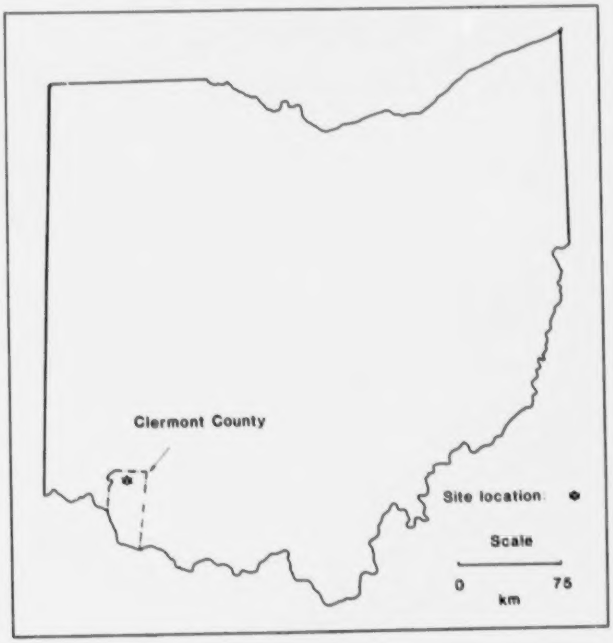


Figure 1. Location of study area in Clermont County, Ohio.

A typical soil profile description for these soils is presented in Appendix A, and a general Avonburg silt loam profile is shown in Figure 3. The basic difference between the Avonburg and Rossmoyne soil types is the depth to mottling. Mottling is present at shallower depths in the Avonburg soils, the somewhat poorly drained member of the catena. In general, the surface layer (0-10 cm) was dark brown to grayish brown silt loam with granular structure. The subsurface horizon (10-25 cm) was brown to yellowish brown silt loam, with dominantly subangular blocky structure. Small manganese nodules were present in this horizon, and in the Avonburg soils mottles were also present. From a depth of approximately 25 to 50 cm was yellowish brown clay loam to silty clay loam, with subangular blocky structure. Mottles and manganese nodules were present in this horizon. A weak fragipan was present from approximately 50 to 125 cm. The fragipan was yellowish brown to brown mottled silty clay loam to clay loam and had prismatic structural units that were smaller than those typically present in a fragipan. Below the fragipan was mottled yellowish brown clay loam with massive structure and many manganese concretions. Interbedded Ordovician limestone and shale bedrock occurred within 3 m over the site and was commonly found at less than 2 m. Many worm and root channels were present throughout the soil profile, and large crayfish channels were abundant over the site.

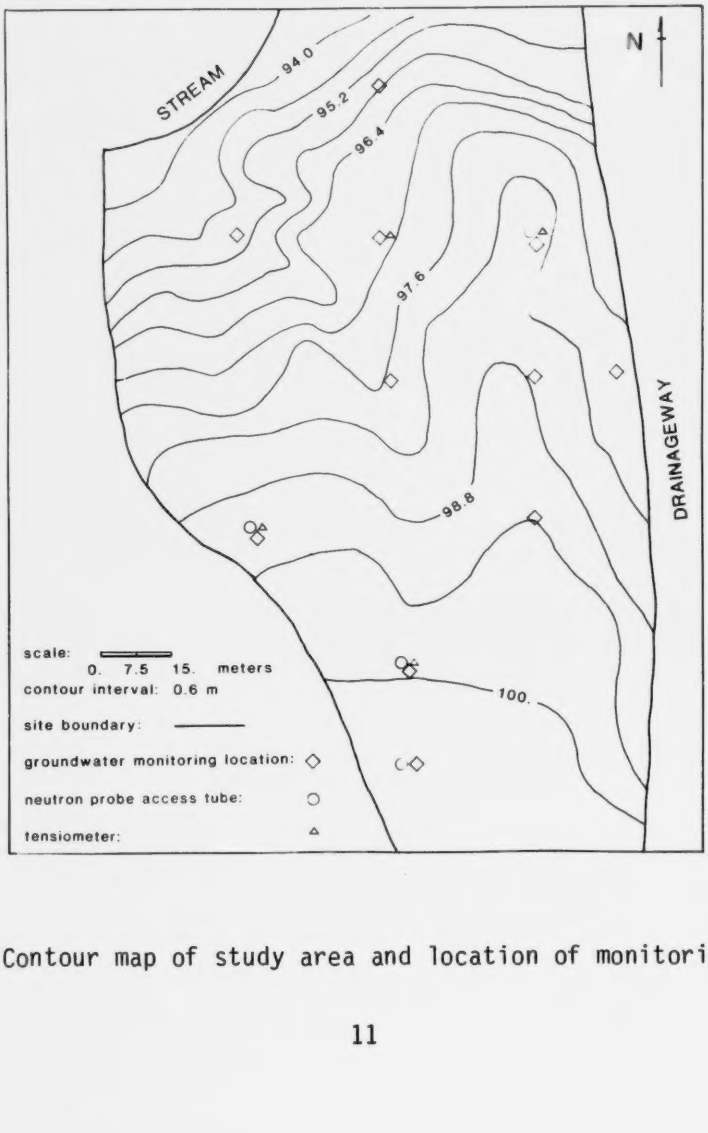

N
STREAM
94.0
95.2
96.4
97.6
98.8
100.
DRAINAGEWAY
scale:
0. 7.5 15. meters
contour interval: 0.6 m
site boundary:
groundwater monitoring location: ◇
neutron probe access tube: ○
tensiometer: △

Figure 2. Contour map of study area and location of monitoring equipment.

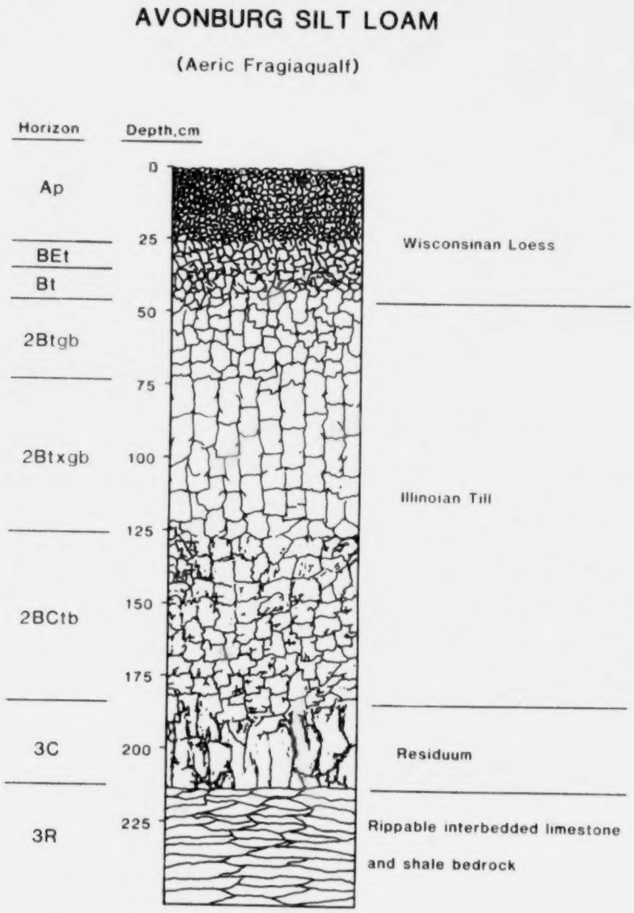

AVONBURG SILT LOAM
(Aeric Fragiaqualf)
Horizon
Depth,cm
Ap
BEt
Bt
2Btgb
2Btxgb
2BCtb
3C
3R
0
25
50
75
100
125
150
175
200
225
Wisconsinan Loess
Illinoian Till
Residuum
Rippable interbedded limestone
and shale bedrock

Figure 3. General soil profile for an Avonburg silt loam.

The ranges of bulk densities in the four soil profiles for typical horizons are presented in Table 2. In most of the soil profiles, the bulk density was highest in the fragipan, which was expected because this horizon is usually the most compact of the developed soil horizons.

Table 3 gives the particle-size analyses of various horizons in a typical soil profile at the study site. The Wisconsinan loess in the surface horizons typically had a silt loam texture. The Illinoian till subsoil generally had a silty clay loam texture, but this varied over the site. Textures of silty clay, clay loam, and clay were also found in the subsoil. No textural difference was found among the other subsoil horizons and the fragipan.

TABLE 2. BULK DENSITIES IN TYPICAL SOIL HORIZONS OF ROSSMOYNE AND AVONBURG SOILS AT THE RESEARCH STATION IN CLERMONT COUNTY, OHIO

Horizon	Depth (cm)	Bulk density, g cm^{-3}
Ap1	0-10	1.2-1.3
Ap2	10-28	1.3-1.5
BEt	28-36	1.4-1.5
Bt	36-48	1.4-1.5
2Btgb	48-69	1.4-1.5
2Btxgb	69-117	1.5-1.7
2BCtxb	117-127+	1.7

TABLE 3. PARTICLE-SIZE ANALYSIS FOR AN AVONBURG SILT LOAM SOIL PROFILE

Depth (cm)	% Sand (2-0.05 mm)*	% Silt (0.05-0.002 mm)	% Clay (<0.002 mm)	USDA Textural class†
0-20	14	70	16	silt loam
43-61	13	48	39	silty clay loam
61-81	13	45	42	silty clay
81-91	7	66	27	silty clay loam
91-102	7	63	30	silty clay loam
102-112	6	62	32	silty clay loam
112-122	4	61	35	silty clay loam
122-132	5	61	34	silty clay loam

* Values in parentheses indicate diameter of particle.
† Soil Survey Staff, 1975.

Several engineering properties of the soils at the field research station were also determined. These included the optimum moisture content for compaction and the Atterberg limits. Soil from a depth of 50 cm was used for both of these tests because this was the depth of the infiltrative surface for most of the onsite wastewater systems studied.

A Standard Proctor compaction test was performed to determine the moisture content at which the soil compacts most readily (Bowles, 1978). This information is used in deciding when to avoid construction, thereby minimizing compaction of the infiltrative surface. Compaction is clearly detrimental to system performance (USEPA, 1980).

The compaction curve for this soil is presented in Figure 4. The optimum moisture content for this standard compactive effort was 20%, by weight. Ninety-five percent of the soil's maximum density occurs between 14.5 and 24.5% moisture.

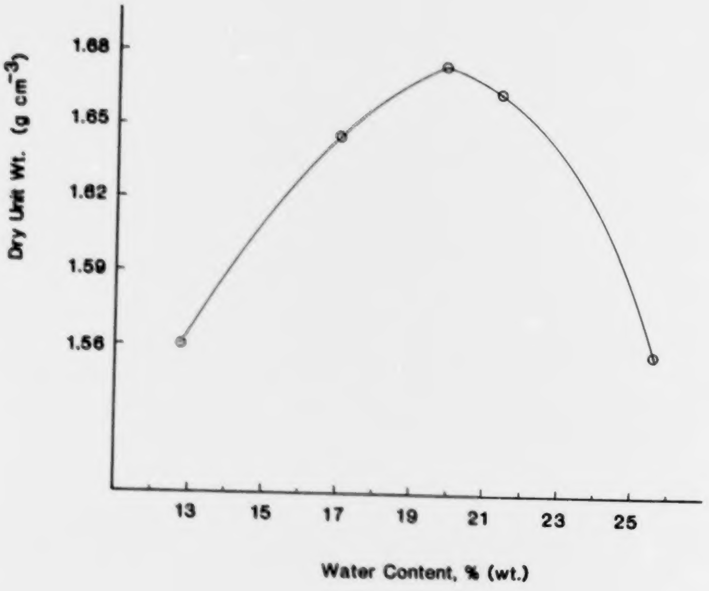


Figure 4. Compaction curve for an Avonburg silt loam soil.

The Atterberg limits are presented in Table 4. These limits show that the soil has medium plasticity--that property of a material that allows it to be deformed rapidly, without rupture, without elastic rebound, and without volume change (Means and Parcher, 1963). The Avonburg silt loam soil in this study can be deformed without a volume change or cracking and will retain its deformed shape between 21 and 49% moisture (by weight). This is typical for a soil with a high clay content (Means and Parcher, 1963). Relating the Atterberg limits to the compaction test, Sowers (1979) found that in clays, the optimum moisture content is often close to or slightly below the plastic limit. This was found to be true in this soil, with the soil having an optimum moisture content of 20% and a plastic limit of 21.

TABLE 4. ATTERBERG LIMITS FOR AN AVONBURG SOIL

Liquid limit	49
Plastic limit	21
Plasticity index	28

Soil Moisture Characteristics

Knowledge of the soil moisture characteristics of a soil is important in determining the ability of the soil to treat and dispose of wastewater. Soil is a three-phase system consisting of solids, liquids, and gases. The movement of liquids and gases in soils occurs in the soil pores. Large soil pores are referred to as macropores, and small pores are termed capillary pores, or micropores. The capacity of soil to conduct water is determined by the size, continuity, and tortuosity of these pores (USEPA, 1980).

The movement of water in soils is determined by the relative energy status, or soil moisture potential, of the soil water. Water flows from points of higher energy to points of lower energy. Two of the most important components of the soil moisture potential are the gravitational potential, which results from the attraction of water toward the center of the earth, and the matric potential, which results from capillary and adsorptive forces associated with the soil matrix, that attract and bind water in the soil (Hillel, 1980a; USEPA, 1980).

When a soil is saturated, all the pores are filled with water, and water moves mainly by gravitational forces. When the soil is unsaturated, only the micropores, which can hold water against the force of gravity, are filled. The smaller the pore, the greater its ability to hold water against gravity. Therefore, as the amount of available water decreases, the diameter of water-filled pores decreases.

Because the water in the micropores in an unsaturated soil is held against the force of gravity, its pressure is less than atmospheric pressure. Thus, the matric potential is actually a negative pressure potential and is often referred to as soil suction or soil moisture tension; the three terms are used synonymously. Matric potential is expressed as a negative number, while suction and tension are positive numbers, since the terms themselves imply negative pressures. Soil moisture tension is zero when a soil is saturated. Increasing suction or tension is associated with soil drying. A thorough discussion of soil water can be found in Hillel (1980a), Bouma (1977), and Bouma et al. (1974).

Wetness and soil moisture tension are functionally related to each other, and the graphic representation of this relationship is termed the soil moisture characterization curve (Hillel, 1980a). The soil moisture characteristic curves for three depths from one of the Avonburg silt loam soil profiles are shown in Figure 5. The curve for the soil at 25 to 36 cm shows that the soil had approximately 48% moisture by volume at saturation, indicating a porosity of 48%, and released 7% moisture between saturation and a tension of 100 cm. The soil at the deeper depths had lower porosities as a result of greater bulk densities and therefore only contained approximately 44.5% moisture by volume at saturation. Only 4.5% moisture was released between 0 and 100 cm of tension at the 48 to 81 cm depth, and 6.4% in the horizon from 81 to 109 cm.

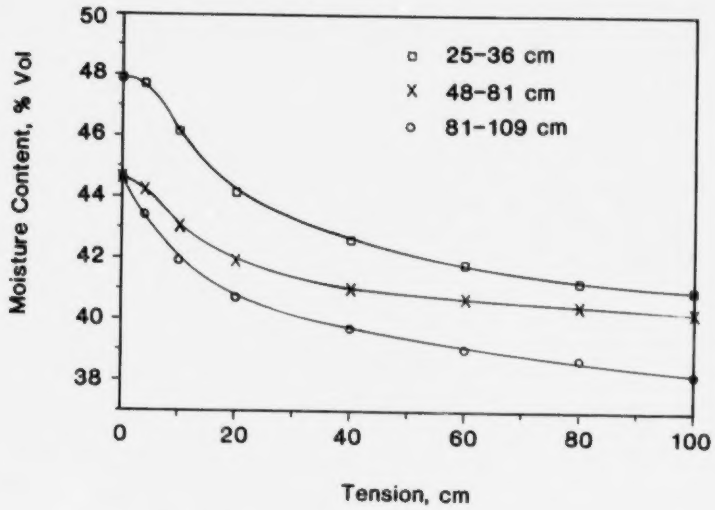


Figure 5. Soil moisture characteristic curves for an Avonburg silt loam soil.

In all of the soil moisture characterization curves developed in this study, most of the total moisture released was removed between 0 and 50 cm of tension. Most of the water removed was probably from the macropores and larger micropores. Because flow rates beyond tensions of 100 cm are considered marginal for effluent disposal (Bouma et al., 1974), the soils in this study only had a limited percentage of pore space involved in the transmission of wastewater; the moisture retention curves indicate that once the macropores have drained, very little flow occurs in these soils.

Shallow Groundwater Characteristics

A high groundwater table occurs in many of the soils in southwestern Ohio during all or part of the year. Because approximately 90 cm of unsaturated soil is desirable between the infiltrative surface of a soil absorption system and a seasonally high water table to ensure adequate treatment of the wastewater before it reaches the groundwater (USEPA, 1978), this condition greatly limits these soils for use as wastewater absorption fields. Therefore, in order to select the most suitable system design for these seasonally saturated soils, it is necessary to characterize the frequency and duration of the shallow free water surface.

Direct measurements of the groundwater table, soil moisture content, and soil moisture potential were made at the field research station to describe the soil moisture regime of a typical site in southwestern Ohio. The monitoring locations are shown in Figure 2. The groundwater table was monitored a minimum of once per week and more often after heavy rainfall events for 2.5 years. The soil moisture content was monitored weekly for one or more years with the use of a neutron moisture gauge, and tensiometers were used to monitor the soil moisture potentials as long as temperatures were above freezing. A summary of the findings is presented here. Appendix A contains a complete report of this study.

The monthly average groundwater table elevation for a typical monitoring location is shown in Figure 6. The free water surface followed a sinuous yearly pattern over the site. It was generally high, within 15 to 60 cm of the soil surface, during the late fall, winter, and spring months, and was closest to the soil surface during March and April. With the increased temperatures and onset of plant growth during the late spring, the groundwater table receded, and was greater than 1.5 m, the detection limit of the monitoring wells, during the summer months. Smeck and Ransom (1983) found a similar pattern for Avonburg and Clermont soils in southwestern Ohio.

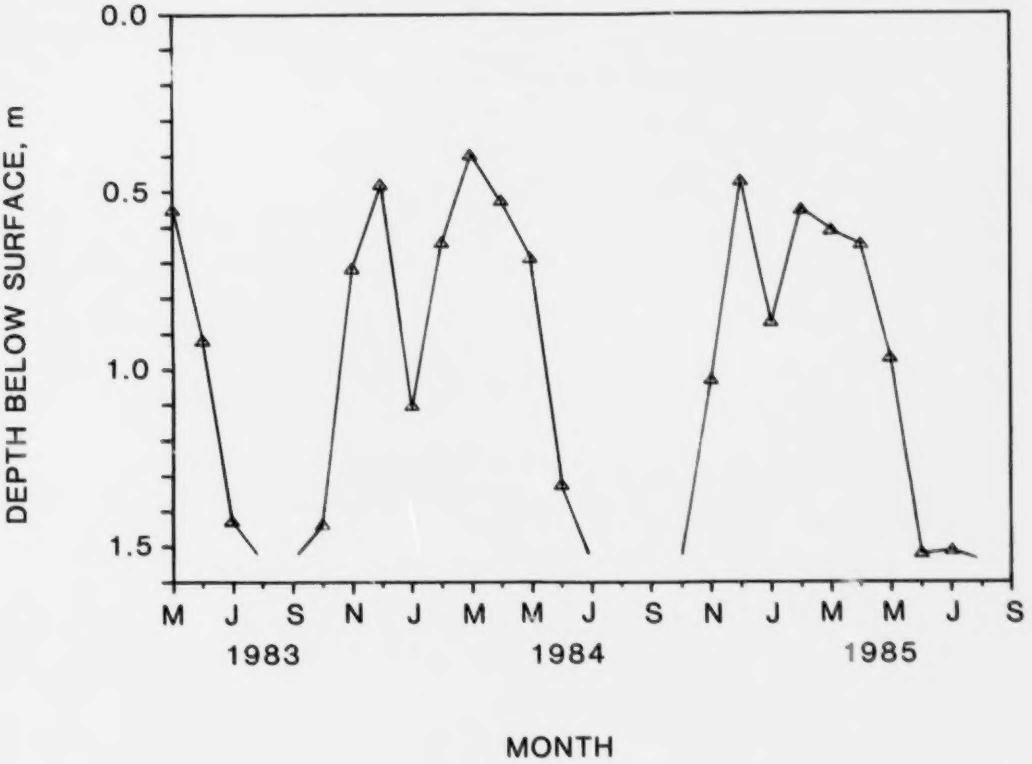


Figure 6. Average monthly groundwater table elevation for a typical monitoring location at the field station.

A perched water table was observed above the water table on several occasions at a few monitoring locations, but only for periods of several days. It appeared that the fragipan at the site did not greatly impede the downward movement of water in these soils.

The soil moisture content, soil moisture potential, and the groundwater table depth at a typical monitoring location are shown in Figure 7 for a time period that includes the winter season through the beginning of groundwater recharge during the following fall for a year with average temperatures and rainfall. The groundwater table was closest to the soil surface during the winter and spring and gradually declined during the spring discharge period (Figure 7c). It was then near or below the detectable limit of the well throughout the summer months, except for the end of July, when a period of heavy rainfall resulted in a groundwater table rise. A rise was again seen in the late fall, during groundwater recharge.

The soil moisture content, presented as the percent of saturation, typically remained relatively constant during the winter and spring seasons and decreased in the upper horizons of the soil profile during the summer months (Figure 7a). Because the neutron scattering method actually measures an average water content over a large sphere of influence, it must be remembered that a value presented for a particular depth, as shown in Figure 7a, is actually the average of as much as a 15- to 60-cm sphere around the point being measured, depending on the soil moisture content at the time of measurement (Gardner, 1965).

The magnitude of the reduction of soil moisture content over the summer months decreased with depth. The greatest decrease occurred in the horizons closest to the soil surface (Figure 7a, 30-cm depth), which was expected because this is the region that is most affected by evapotranspiration. Below approximately 1 m, however, very little change in the soil moisture content occurred over most of the study area, even though the free water surface was below this depth. Only approximately 7% moisture was removed from the 120-cm depth during the summer months, indicating that the soil at this depth was at or near saturation throughout the study. The soil moisture content at all depths again increased during the fall groundwater recharge period.

The soil matric potential also suggested that the soils remained relatively wet in the lower horizons throughout the year (Figure 7b). During the summer months the soil matric potential in the upper horizons of the soil profile was highly negative, indicating unsaturated conditions, with the greatest matric potential occurring near the soil surface, and often exceeding -800 cm, the practical monitoring limit of a tensiometer (Richards, 1965). The matric potential increased in the lower horizons, but only reached a maximum of about -100 cm at the 107-cm depth. The water lost in this horizon over this tension range (approximately 7%) correlates with the amount of water lost in the moisture retention curve over the same tension range for a similar depth (Figure 5, 81-109 cm).

During this study, the soil moisture potentials occasionally indicated saturated conditions (positive pressure potentials) for several weeks after the groundwater table receded below the depth of the tensiometer (Figure 7b, 6/8 to 7/8). This usually occurred only at depths greater than 75 cm. The reason for this unusual reaction lag is not known.

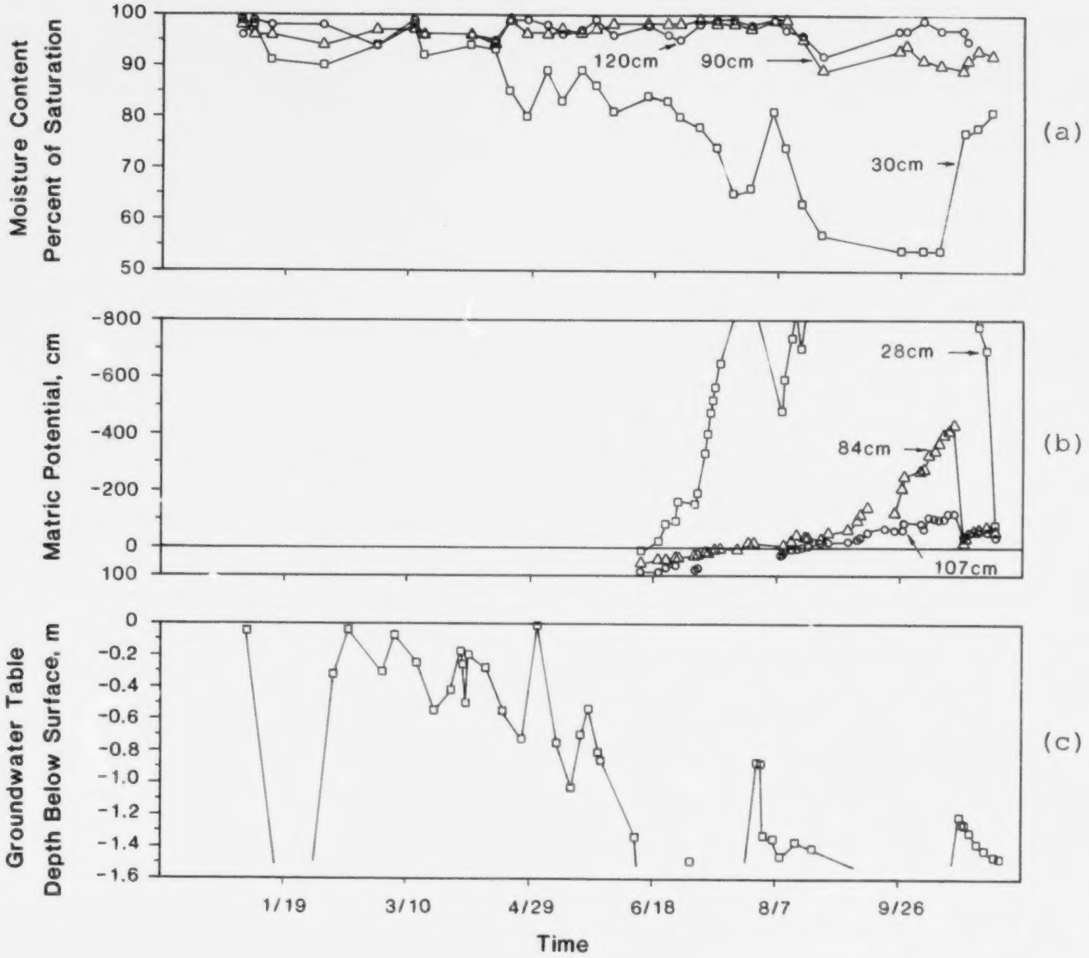


Figure 7. Soil moisture content (a), soil moisture potential (b), and groundwater table elevation (c) for a typical monitoring location in an Avonburg silt loam.

The soil moisture content and soil moisture potential data suggest that the Illinoian till soils in this study were capable of retaining a high percentage of water, even after the observed free water surface receded below the detection limits of the wells, and were saturated or nearly saturated in the lower horizons throughout most of the year. For uses such as onsite wastewater disposal, this near-saturated condition can be just as prohibitive as the presence of a water table.

Others have found little change occurring in the soil moisture content in lower horizons of soils with similar textures. Harlan and Franzmeier (1974) found that the volumetric moisture content of the 2C horizon of a

Crosby silt loam soil (Aeric Ochraqualf, fine, mixed, mesic) varied only 7% over a growing season, indicating that this horizon was always close to saturation. Vepraskas et al. (1974) found that soil moisture tensions only reached 20 and 30 cm in the B31g horizon of a Grays silt loam soil (Mollic Hapludalf), which corresponded to relatively high moisture contents. They assumed that in this range only the larger pores such as channels and planar voids were filled with air, while the fine pores in the peds remained filled with water.

The observed groundwater table in this study continually fluctuated with rainfall events during the fall, winter, and spring. The largest fluctuations were in the nearly level landscape positions during the late spring and fall, when the groundwater was discharging or recharging, respectively. Water table rises of more than 1 m within several days after a major rainfall event were common (Appendix A).

Other studies have reported that shallow water tables frequently show a disproportionate response to infiltration (Gillham, 1984). The responses that have been shown to contribute to these observed water table rises include vertical infiltration of rainfall, net lateral inflow from upslope, and air pressure buildup in the soil (Fayer and Hillel, 1982). It is likely that all of these processes contributed in some degree to the wide fluctuations in the observed water table in this study.

During years with typical weather conditions, the groundwater table in the nearly level upslope landscape positions, which were typically Avonburg soils, remained nearest to the soil surface longer in the spring and rose to high levels sooner in the fall than the soils in the gently sloping to sloping downslope positions, which were predominantly Rossmoyne soils. This variation most likely resulted from soil moisture content differences that are due to landscape position. The soils in the nearly level upslope positions can remain saturated longer in the spring and can become saturated sooner in the fall because little runoff occurs in nearly level landscape positions, and most of the rainfall that falls on these areas can infiltrate and percolate through the soil. Also, because of the low slope gradient, less lateral flow that would remove water within the soil profile occurs. However, in the sloping, downslope positions, precipitation can run off the site and less water infiltrates into the soil. Therefore, the soils in these areas will not become saturated as soon as those in the upslope positions. Also, water could be removed internally from these soils by lateral flow as a result of the steeper slope gradient at these slopes.

SUMMARY

This study showed that soils such as those typically found in southwestern Ohio can retain large volumes of water in the lower horizons throughout the year, even when a seasonally high groundwater table is low or absent. This nearly saturated to saturated condition cannot be predicted with groundwater table data alone. Soil moisture content or soil moisture tension data are also required to understand the soil moisture regime of these soils. Monitoring at close spacings and at frequent intervals may also be required to accurately describe the soil moisture regime during various seasons and in different landscape positions.

The soil and site characteristics determined in this study indicate that the majority of the soils in southwestern Ohio have severe limitations for onsite wastewater diposal. Because the cost of public sewer systems is prohibitive in most sparsely populated areas, alternative onsite treatment and disposal systems are needed to overcome soil limitations such as slow permeability and wetness.

SECTION 5

ALTERNATIVE SYSTEMS THAT RELY ON THE SOIL FOR TREATMENT AND DISPOSAL OF WASTEWATER

INTRODUCTION

Subsurface soil absorption systems have been used for many years to treat and dispose of domestic wastewater in areas not served by centralized sewer systems. Soil absorption and treatment is desirable because of the large natural capacity of the soil to break down organic and other chemical substances as well as remove pathogenic organisms and viruses. Also, a properly designed, constructed, and maintained subsurface soil absorption system can perform reliably over a long period of time with little attention and minimal costs (USEPA, 1980). Therefore, subsurface absorption is the preferred method of onsite wastewater disposal.

The conventional septic tank/soil absorption system is the most commonly installed subsurface soil absorption system. The successful operation of this system requires specific soil and site characteristics such as a sufficiently large lot size with a slope less than 10%, an area outside of the flood plain, 60 to 120 cm of suitable soil between the infiltrative surface and the groundwater table, bedrock, or other restrictive layer, and a permeability that allows wastewater to move readily through the soil profile, while flowing slowly enough for treatment to occur (USEPA, 1980).

Because of these specific requirements, many soils in the United States, including the majority of soils in southwestern Ohio, are not suitable for the installation of conventional systems. Therefore, alternative systems have been developed over the past several decades to overcome the limitations of soils and sites that cannot accommodate conventional systems. Most of these alternative systems have been adequately reviewed (USEPA, 1978, 1980; Ronayne et al., 1982).

Several alternative systems that rely on the soil for treatment and disposal were installed and evaluated in this study. The systems included a full-scale alternating field system, scaled-down low-pressure pipe, gravelless, and shallow trench systems, and laboratory-scale columns that represented a mound system. These studies are summarized in this section.

ALTERNATING SOIL ABSORPTION FIELD SYSTEM

Continuous application of septic tank effluent to subsurface wastewater treatment and disposal systems results in the development of a clogging zone at the gravel/soil interface (Otis, 1984). This clogging zone restricts the rate of wastewater infiltration into the soil and can become so restrictive that the system is unable to accept the volume of wastewater it receives. As a result, sewage backs up or seeps out onto the ground surface. Most properly designed, installed, and maintained systems that depend upon continuous inundation of the soil surface ultimately fail as the result of this clogging mat (McGauhey & Winneberger, 1964). Clogging mat formation has been thoroughly reviewed (McGauhey & Winneberger, 1964; Otis, 1984; USEPA, 1978).

Solutions for a failed absorption system have generally been costly and usually consist of replacing the failed system or rebuilding the existing system after removing the clogging layer (Bishop & Logsdon, 1981). An effective method of rejuvenating a failed system without replacing it is to take it out of service for a prolonged period of time and rest it (USEPA, 1980). Resting allows the field to drain gradually, exposing the clogged infiltrative surface to air and creating aerobic conditions. This hastens decomposition of the clogging materials, restoring much of the infiltrative capacity of the soil (Winneberger et al., 1960).

Rejuvenation may also be accomplished by alternating effluent disposal between two soil absorption fields. One field is used while the other field is "rested," allowing its infiltrative capacity to be restored.

An alternating soil absorption field system was installed at a private residence in Hamilton County, Ohio, where an existing conventional system had failed after 18 years of satisfactory operation. Because no outlet was available for a surface water discharge system, and because the area available for a replacement system was small, the only reasonable option at this site was to install additional soil absorption trench. The amount of trench that was installed was comparable to the amount in the existing system, which had apparently been satisfactory for 18 years even though it was grossly undersized for the three-bedroom home. Because the two areas of trench were of similar size, effluent was alternated between them in an effort to restore the infiltrative surface in the existing system.

The primary objective of this project was to correct a failure at a private residence. A secondary objective was to determine if the failed existing system could be rehabilitated through resting, and to determine if the alternating soil absorption field is an effective alternative system for use in southwestern Ohio. A summary of this study appears here. A complete report is presented in Appendix B.

The existing system at the site, located in Figure 8, had 38 m of absorption trench, spaced into three trenches that were installed across a 10% slope. In the northern section of this system, bedrock occurred within 80 cm, and the trenches in this area were only separated from the bedrock by several centimeters of soil. The newly installed system, referred to as the alternating system (Figure 8), was installed in the fall of 1982 and consisted of 32 m of trench divided into two main trenches, referred to as the upper trench (upslope) and the lower trench (downslope). It was installed on a 7%, north-facing sideslope.

The original topsoil in the location of the alternating system was covered with 0 to 1 m of clay and clay loam fill material. The buried topsoil was approximately 20 cm of dark grayish brown silt loam. The subsoil was dark yellowish brown silty clay loam and clay loam and was mottled throughout, indicating the presence of a fluctuating high water table. Interbedded limestone and shale bedrock was present at approximately 180 cm. The soil had percolation rates ranging from 3 to 32 min/cm at a depth of approximately 50 cm.

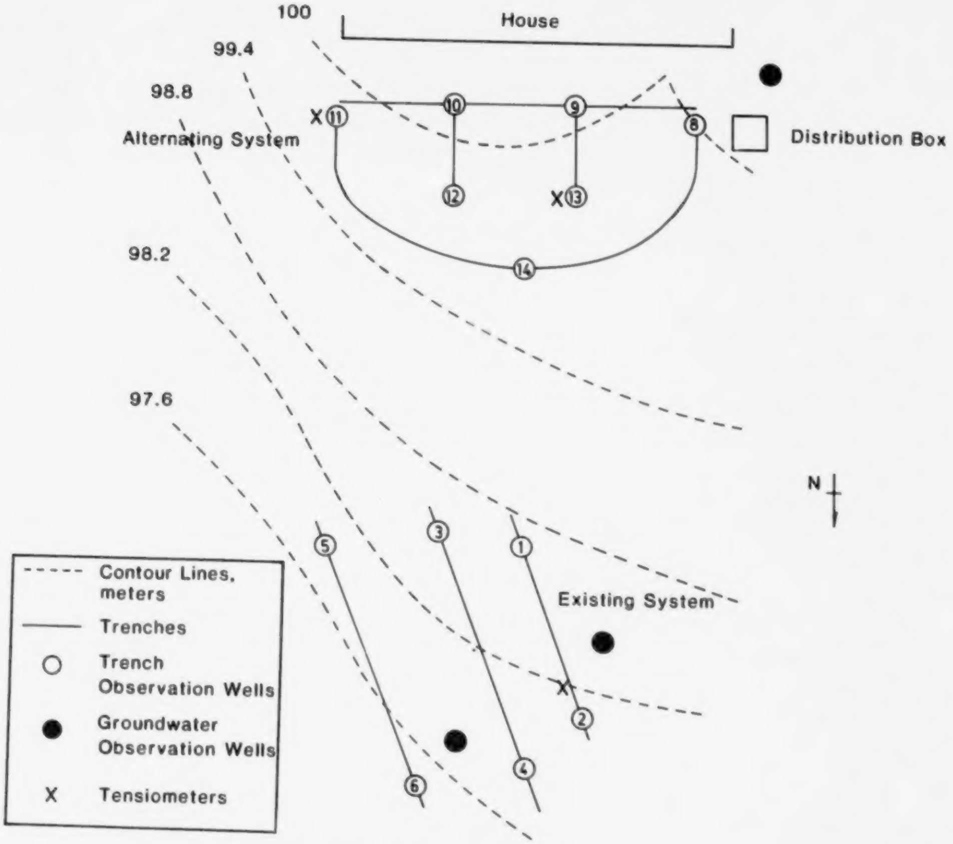

100
House
99.4
98.8
Alternating System
Distribution Box
98.2
97.6
N
Existing System
X
1
2
3
4
5
6
8
9
10
11
12
13
14
Contour Lines, meters
Trenches
Trench Observation Wells
Groundwater Observation Wells
Tensiometers

Figure 8. Location of the existing system, alternating system, and monitoring devices.

Tensiometers and observation wells were installed in both the existing and the alternating systems to monitor the soil moisture tension in the soil surrounding the trenches and the ponding depths in the trenches, respectively. The locations of the monitoring devices are shown in Figure 8. Three additional observation wells were installed outside of the trenches to determine the depth to the fluctuating high water table (Figure 8, Wells 0, 7, 15).

Ponding depth measurements were made weekly during the 3 years of the study. The tensiometers were read weekly only during those times of the year when they are operative, usually May to November. (Tensiometers cannot be used in freezing temperatures.) Rainfall measurements were made weekly using a tipping bucket rain gauge. The effluent was applied to the trenches by a pump that was installed in a distribution box. The pump was designed to deliver effluent into the trenches when the amount of effluent in the distribution box reached a certain height. Throughout the study, an average of 700 l of wastewater was applied daily, as determined by water meter readings. Valves were placed in the distribution box to control the volume of effluent directed to the two systems.

It was initially planned that the existing system would be kept out of service for at least one year to restore the infiltrative surface. Alternation between the two systems would then be scheduled for 6-month intervals. Unforeseen problems occurred, however, and system alternation was performed as shown in Table 5.

TABLE 5. OPERATION SCHEDULE FOR THE ALTERNATING SOIL ABSORPTION FIELD SYSTEM

Phase of operation	Date	Days in service*	System in operation
I	9/16/82-5/26/83	1-253	Existing system
II	5/26/83-11/16/83	253-427	Alternating system
III	11/16/83-2/20/84	427-523	Existing system
IV	2/20/84-6/13/84	523-637	Alternating system
	6/13/84-7/20/84	637-674	Alternating system--lower trench only
	7/20/84-12/21/84	674-828	Alternating system--lower and upper trenches
V	12/21/84-9/3/85	828-1084	Existing and alternating systems

* The days that the systems were in service beginning with the time that monitoring was initiated.

Phase I

During Phase I (9/16/82 to 5/26/83), wastewater was applied only to the existing system to establish baseline ponding depths. The trenches were filled to their maximum capacities during most of this period. Although no effluent was applied to the alternating system, it appears that some effluent did enter one of the trenches as the result of a cracked distribution box.

Phase II

Effluent was diverted to the alternating system in May 1983. In the existing system, the ponding depth gradually decreased and eventually dried out after approximately 2 months (Figure 9, Well 4). Water again ponded in this system in the fall, as the result of a high groundwater table.

In the alternating system, the ponding depth gradually increased (Figure 9, Wells 11, 13). However, the ponding depth in the lower trench (Well 11) had a greater increase than the upper trench (Well 13) as the result of an effluent distribution problem.

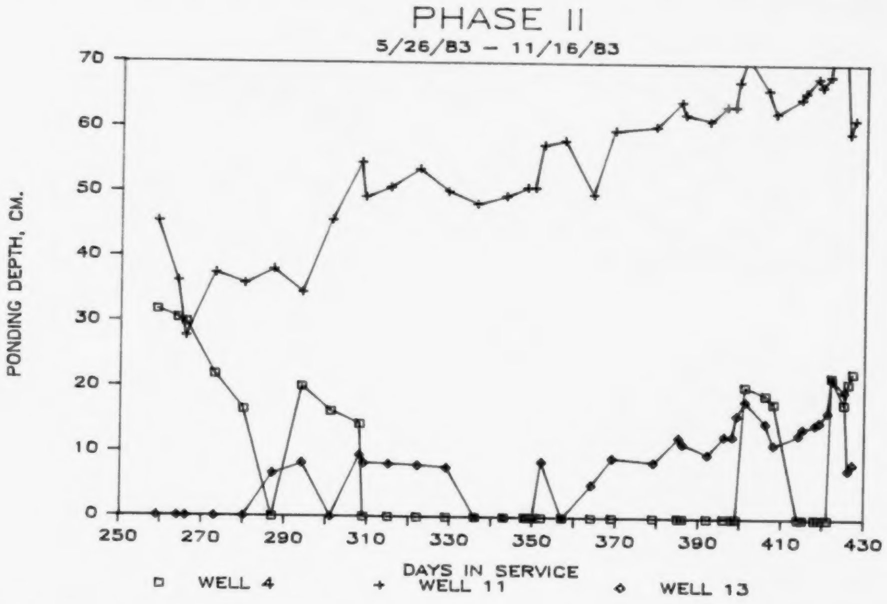


Figure 9. The effluent ponding depth in the trenches in the existing and alternating systems during Phase II.

Phase III

During Phase III, wastewater was diverted to the existing system to correct the distribution problem between the upper and lower trenches in the alternating system. The ponding depth in the existing system was already high as the result of groundwater intrusion, and little change occurred in the ponding depth with the addition of effluent (Figure 10, Well 4). The trenches in the alternating system quickly dried out and remained dry throughout Phase III (Figure 10, Wells 11, 13).

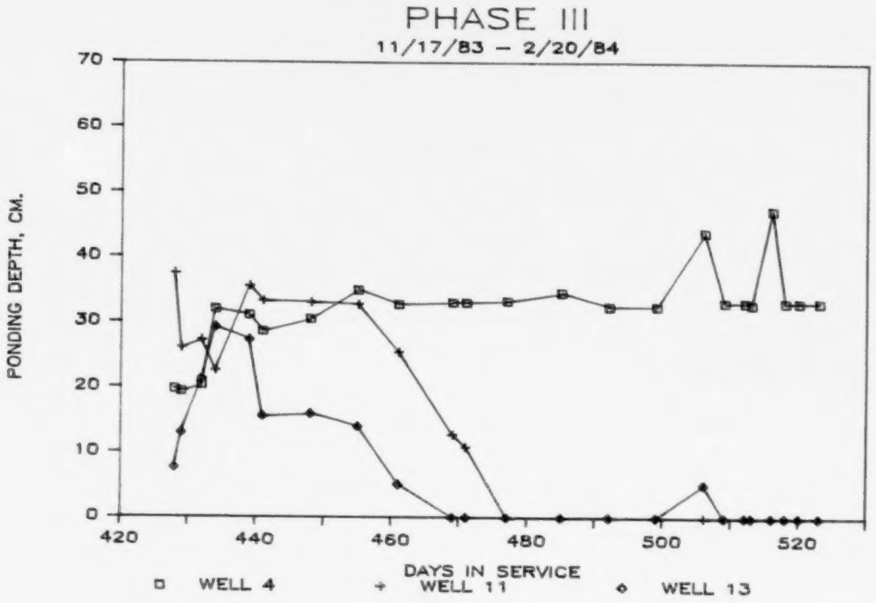


Figure 10. The effluent ponding depth in the trenches in the existing and alternating systems during Phase III.

Phase IV

Effluent was diverted back to the alternating system in February 1984. Although no effluent was being supplied to the existing system during this period, the ponding depth remained high in this system until spring, and then slowly decreased through June (Figure 11). The trenches in the existing system remained dry for approximately 60 days, and as in the previous year, were again inundated with groundwater during the fall.

In the alternating system, the ponding depth in the trenches quickly increased after effluent was redirected to it (Figure 12). Large fluctuations in the ponding depths then occurred with rainfall events and with valve adjustments that were made to regulate the volume of effluent directed to each trench.

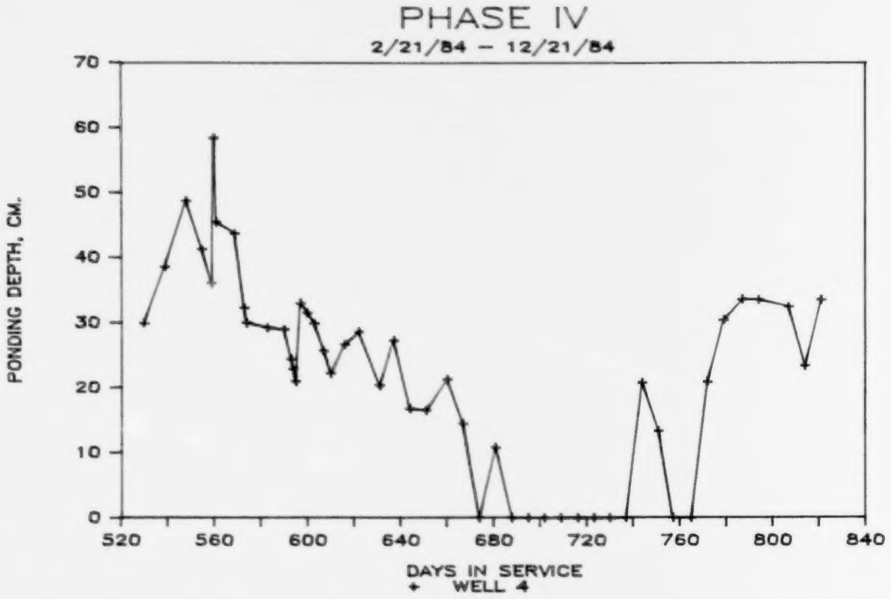


Figure 11. The effluent ponding depth in the existing systems during Phase IV.

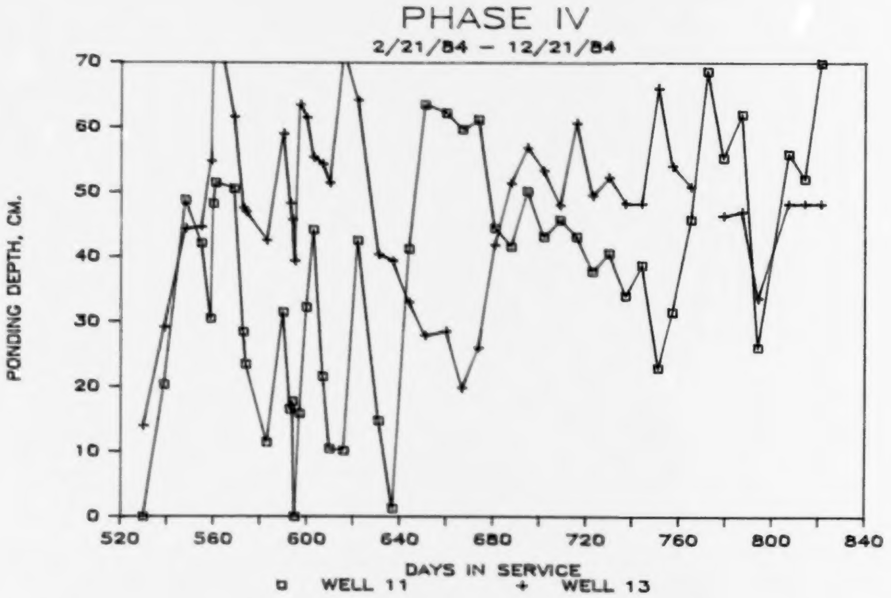


Figure 12. The effluent ponding depth in the alternating system during Phase IV.

Phase V

During the second winter of operation of the alternating system, surfacing of effluent occurred, indicating that additional trench capacity was required to dispose of the wastewater. Therefore, effluent was directed to both the existing and the alternating systems during December 1984. Both systems were then required to treat and dispose of the effluent produced throughout the remainder of the study. Constant fluctuations occurred in the ponding depths because valves were used to regulate the volume of flow to each system.

During June 1985, the entire volume of effluent was directed to the existing system to determine whether this system had rejuvenated sufficiently to dispose of all of the wastewater. Effluent surfaced within 3 weeks, indicating that the system had not been sufficiently rejuvenated during the study.

Precipitation Effects

The ponding depths in the trenches and the elevation of the groundwater table fluctuated with rainfall events throughout the study during the late fall, winter, and spring, when soil moisture contents were high and evapotranspiration rates were low. The rise in the groundwater table occurred rapidly after a rainfall event and then decreased rapidly. This groundwater often rose above the depth of the trenches and actually inundated the trenches in the existing system, resulting in ponding even when effluent was not applied to this system, as was seen in Figures 9 and 11, Well 4. Groundwater did not appear to inundate the trenches in the alternating system, however.

The groundwater intrusion in the existing system was most likely the result of the close proximity of the trenches to the bedrock. Approximately 1 m of soil is recommended between the infiltrative surface of the trench and bedrock (USEPA, 1980) because groundwater can back up, or perch, above the nonporous bedrock. The trenches in the existing system were too close to the bedrock and were consequently inundated with the perched groundwater. In the alternating system, however, sufficient soil was present between the trenches and the bedrock, and inundation with groundwater was not observed.

Groundwater intrusion in the trenches of the existing system most likely prevented the total rejuvenation of the infiltrative surface. Biochemically reduced conditions prevail when groundwater or effluent is present in the trenches, and oxidation of the clogging mat cannot occur. The trenches in the existing system were dry for only a maximum time of 90 continuous days, and evidently this was not enough time to break down the clogging mat. The importance of maintaining a suitable isolation distance between the trench bottom and an impervious horizon is clearly seen in this study.

This study showed that after intermittent periods of resting lasting no more than 90 days, rejuvenation of the existing system was insufficient to enable it to treat and dispose of the entire volume of wastewater produced at the site. However, the addition of 32 m of trench effectively prevented effluent surfacing during the period of this study.

LOW-PRESSURE PIPE SOIL ABSORPTION FIELD

The low-pressure pipe (LPP) soil absorption field has been found to be an acceptable alternative onsite wastewater disposal system in North Carolina (Carlile, 1979; Cogger & Carlile, 1984; Floyd, 1982; Hargett, 1984). This system was designed and tested in North Carolina for use in sites and soils that do not meet the requirements for a conventional septic system (Carlile, 1979). It was developed to provide aerobic treatment in soils with high water tables or shallow restrictive horizons, using shallow trenches with dosing and resting cycles designed to maintain aerobic conditions (Cogger & Carlile, 1984).

The site and soil requirements for the LPP system, as developed by the Triangle J. Council of Governments (1979), are a minimum soil depth of 76 cm from the finished soil surface to any underlying restrictive horizon such as consolidated bedrock or hardpan, or to the seasonally high water table. At least 30 cm of good aerobic soil material must be at the surface of the total 76-cm soil depth. Sites with slopes greater than 10% are not suitable for LPP systems.

The LPP system consists of a network of small diameter perforated pipes placed 15 to 45 cm deep in narrow trenches 10 to 15 cm wide. Septic tank effluent is pumped through the LPP system in controlled doses to ensure uniform distribution throughout the entire absorption field (Carlile, 1979). The system is designed to produce even distribution of the effluent, maintain maximum separation of the distribution lines above water tables, and control dosing of effluent throughout the soil absorption area for optimum dosing and resting cycles.

An LPP system was installed at the field research station located in Clermont County, Ohio (Figure 1), to determine whether it was an acceptable alternative onsite wastewater disposal system in a colder climate than North Carolina, and in a location that did not meet the site and soil suitability criteria established by the Triangle J. Council of Governments (1979). It was installed on a 10%, east-facing side slope in a moderately eroded Rossmoyne silt loam soil (Aquic Fragiudalf, fine-silty, mixed, mesic). A fluctuating high water table was present at this site at a depth of approximately 26 cm. (The Triangle J. Council of Governments (1979) recommends a minimum soil depth of 76 cm from the finished surface to the seasonally high water table; 10% is also the maximum slope recommended by the Council.) A summary of this report appears here. A complete report of this study is presented in Appendix C.

The LPP absorption field installed in this study consisted of 5 trenches that were 12.2 m long and spaced 1.5 m on center, as shown in Figure 13. The trenches were 20 cm wide and approximately 28 cm deep and were installed on the contours with the use of a trenching machine. A gravel layer 18 cm thick was placed in the bottom of the trench. A 3.2 cm diameter plastic distribution pipe was placed on top of this and was covered with a thin layer of gravel. A synthetic fabric was laid above the gravel and the entire trench was covered with 7.6 cm of soil to bring the trench up to grade. An additional 10 cm of soil was mounded above the trench to prevent surface water from ponding and to allow for future settling.

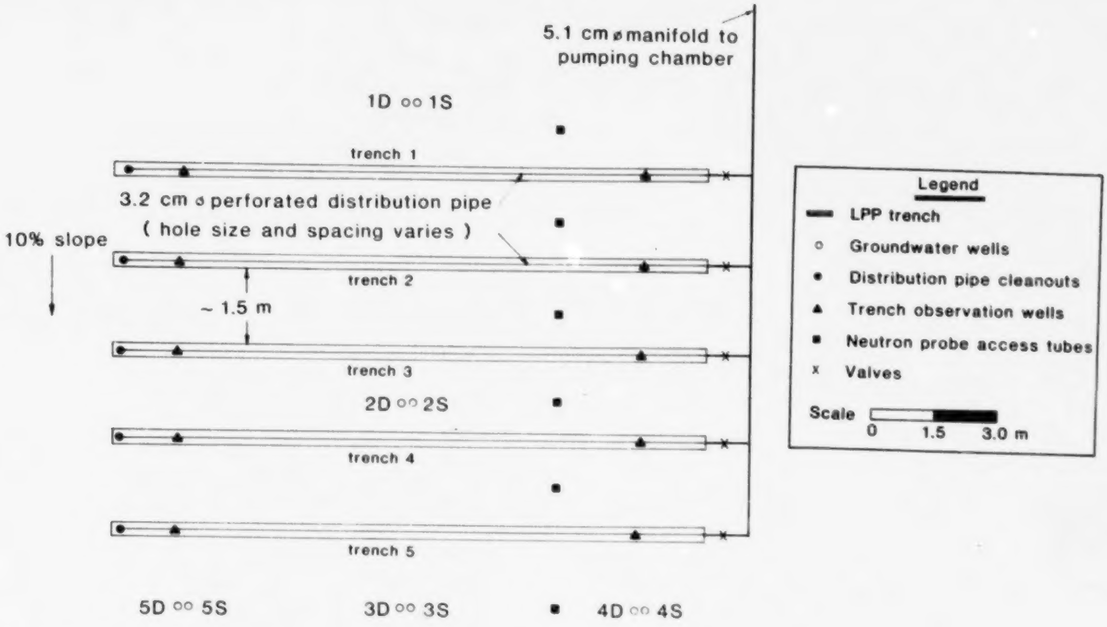

5.1 cm ø manifold to pumping chamber
1D oo 1S
trench 1
3.2 cm ø perforated distribution pipe
(hole size and spacing varies)
10% slope
trench 2
~ 1.5 m
trench 3
2D oo 2S
trench 4
trench 5
5D oo 5S
3D oo 3S
4D oo 4S
Legend
LPP trench
Groundwater wells
Distribution pipe cleanouts
Trench observation wells
Neutron probe access tubes
Valves
Scale
0
1.5
3.0 m

Figure 13. Diagram of the low-pressure pipe soil absorption field and monitoring equipment locations.

Pressure distribution of septic tank effluent was employed using design criteria presented by Otis (1981). The pressure distribution system used a 3785-l pumping chamber equipped with a submersible pump activated by an electronic timer. Effluent was initially applied in a single daily dose of approximately 300 l. Because the loading rate in these systems is based on the total surface area that the system covers, this dose was equivalent to a loading rate of 4.1 lpd/sq m. The loading rate was later adjusted to comply with the seasonal changes in the system's ability to absorb effluent, as determined by the depth of effluent ponding in the trenches.

Observation wells installed in the trenches and in the natural soil adjacent to the system (Figure 13) were used to monitor the ponding depth in the trenches and the depth to the groundwater table, respectively. The groundwater observation wells were installed in pairs at depths of approximately 60 and 145 cm.

The average concentrations of specific constituents of septic tank effluent applied to the LPP system are presented in Table 6. In general, the concentrations of the parameters tested were slightly lower than concentrations reported in the literature for septic tank effluent (Otis and Boyle, 1976).

TABLE 6. CONCENTRATION OF THE SEPTIC TANK EFFLUENT APPLIED TO THE LOW-PRESSURE PIPE SOIL ABSORPTION FIELD

Parameter	Number of samples	Mean	Median	Range
		mg/l		
NH_3-N	58	16.5	17.2	2.9-30.0
NO_2+NO_3-N	58*	0.1	0.1	<0.1-0.9
TKN	58	20.1	20.6	3.1-44.1
Total-P	58	7.6	8.4	0.9-17.1
COD	58	86.0	86.0	21.0-218.0
Chloride	74	75.9	61.2	5.0-497.0

* Thirty samples had a concentration less than the detectable limit of 0.1 mg/l. The mean and median was calculated by assuming a value of 0.1 mg/l for these samples.

The loading rate history for the LPP system in this study is given in Table 7. Effluent absorption was variable and dependent on environmental factors. The loading rates ranged from 4 to 10 lpd/sq m, with the highest rate occurring during the first summer of operation. After the first year, the loading rate was kept constant at approximately 4 lpd/sq m throughout all of the seasons, although occasional periods of failure occurred. It appeared that a long-term loading rate of 4 lpd/sq m or lower was reasonable throughout most of the year for the soil and site conditions in this study.

TABLE 7. LOW-PRESSURE PIPE SOIL ABSORPTION FIELD LOADING RATE HISTORY

Time period	Peak flow rate, lpd	Peak loading rate, lpd/sq m
Sep 1983 - Apr 1984*	303	4.1†
Apr - May 1984§	454	6.1
May - Jul 1984§	568	7.7
Jul - Sep 1984§	757	10.2
Sep - Oct 1984	568	7.7
Oct 1984 - Aug 1985	303	4.1

* The system occasionally failed during this time period when the seasonal water table was near the ground surface.

† Based on an absorption area of 74 sq m.

§ Electrical malfunctions occurred during this time period, and the LPP system was occasionally taken out of service.

Effluent Distribution

Effluent was ponded prior to a dosing event in the trenches of the LPP system during the late fall, winter, and spring, and the use of pressure distribution and dosing did not always achieve the goal of resting the trenches between dosing events. During the summer months, a period of resting between dosing events did occur in some of the trenches. Throughout the study the trenches in the downslope positions consistently had the greatest depth of ponding, even during the summer months, although effluent was equally distributed to all of the trenches. This unequal ponding may have resulted from different absorption rates of the soil surrounding the trenches, or from the addition of effluent from the upslope trenches by lateral flow. Hargett (1984) also observed this trend in LPP systems on sloping sites in North Carolina. Therefore, it is recommended that a valve be installed between the manifold and the distribution pipe of each trench so that the individual loading rates can be adjusted. The valves could also be used to take a trench out of service for long-term resting.

Freezing Potential

The LPP soil absorption field was originally developed and used in North Carolina where the winters are mild and the soil rarely freezes to any significant depth for extended periods of time. The frost line is deeper in northern states, however, and freezing of the soil surrounding the trenches may be a potential problem because of their shallow placement. In this study, the system was in continuous operation during only one of the two winters, and no freezing was detected. During the other winter, however, the system was taken out of service in January for repairs, and the soil surrounding the trench froze. The soil thawed before effluent was again applied and the effect of the freezing was not determined. More research on this topic is suggested before specific recommendations can be made concerning the freezing potential of this design.

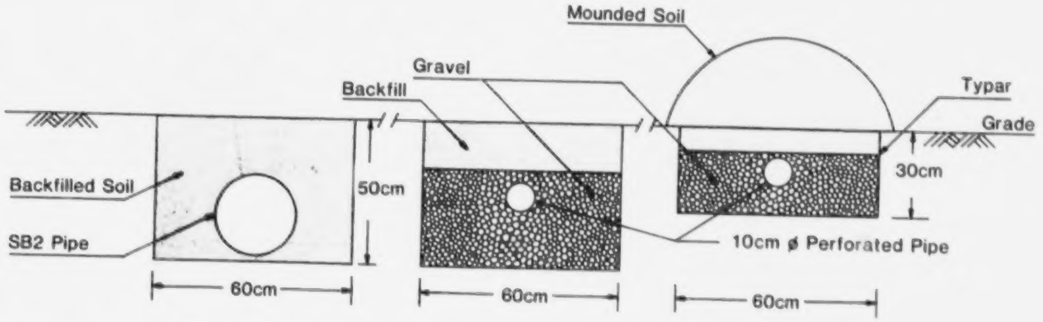
The LPP system in this study appeared to perform adequately at low loading rates during most of the year, with occasional seasonal failure.

ALTERNATIVE TRENCH DESIGNS

Three trench designs were evaluated at the field research site. Two of these were alternative designs and included gravelless trenches designed to reduce costs of installation, promote improved distribution of wastewater in the trench, increase retention of solids in the pipe, and increase capillary movement of wastewater around the soil surface (Carlile and Osborne, 1981), and shallow trenches designed to overcome the shallow groundwater table limitation. The third design was conventional. Specific objectives of this study were to determine which of the three designs absorbed the greatest volume of wastewater, to evaluate the performance of SB2® gravelless distribution tubing, and to determine the suitability of an Avonburg soil under the conditions of this study to absorb, treat, and dispose of septic tank effluent. A summary of this report appears here. A complete report of this study is presented in Appendix D.

The trenches were installed on a nearly level upland landscape position in an Avonburg silt loam soil (Aeric Fragiaqualf) at the field station in southwestern Ohio (Figure 1). Characteristics that limit the soils at this site for use in subsurface wastewater disposal include a fragipan, slow permeability, and a fluctuating high water table (Lerch et al., 1975).

The trench designs in this study, as shown in Figure 14, included the following: conventional trench 50 cm deep and 60 cm wide, filled with 36 cm of standard 2.5- to 5.0-cm diameter washed gravel; gravelless trench that consisted of a 25-cm diameter corrugated plastic tubing covered with a porous wrap manufactured by Advanced Drainage Systems, Inc., and marketed under the brand name of SB2® gravelless leach bed tubing, placed in 50 cm deep and 60 cm wide trenches (Lang, 1981); and a shallow trench that was installed in a manner similar to the conventional trench, but at a depth of 30 cm, and a total gravel depth of 20 cm. An additional 45 cm of soil was mounded above the trench to prevent freezing. The trenches were each 6 m long and were installed in duplicate 6 m apart. The locations of these trenches are shown in Figure 15.



(a) Gravelless Trench

(b) Conventional Shallow Trench

(c) Very Shallow Trench

Figure 14. Cross section of the three trench designs.

Each trench had its own pumping station that consisted of a calibrated pump, check valve, gate valve, magnetic float switch, and time recording device. This allowed for an accurate measurement of the volume of liquid absorbed by each trench. When the depth of ponding in the trench fell to approximately 17 cm as a result of effluent absorption, a magnetic float activated reed switches within the magnetic float switch, which in turn activated the pump, and septic tank effluent was applied to the trench. When the depth of ponding reached a level of approximately 23 cm, the magnetic float activated a second reed switch, turning the pump off. The length of time the pump was running was recorded using digital timers, and because the pumps were calibrated, the volume of effluent absorbed per day was calculated. This mechanism also maintained a relatively constant ponding depth in the trenches and allowed for the application of effluent

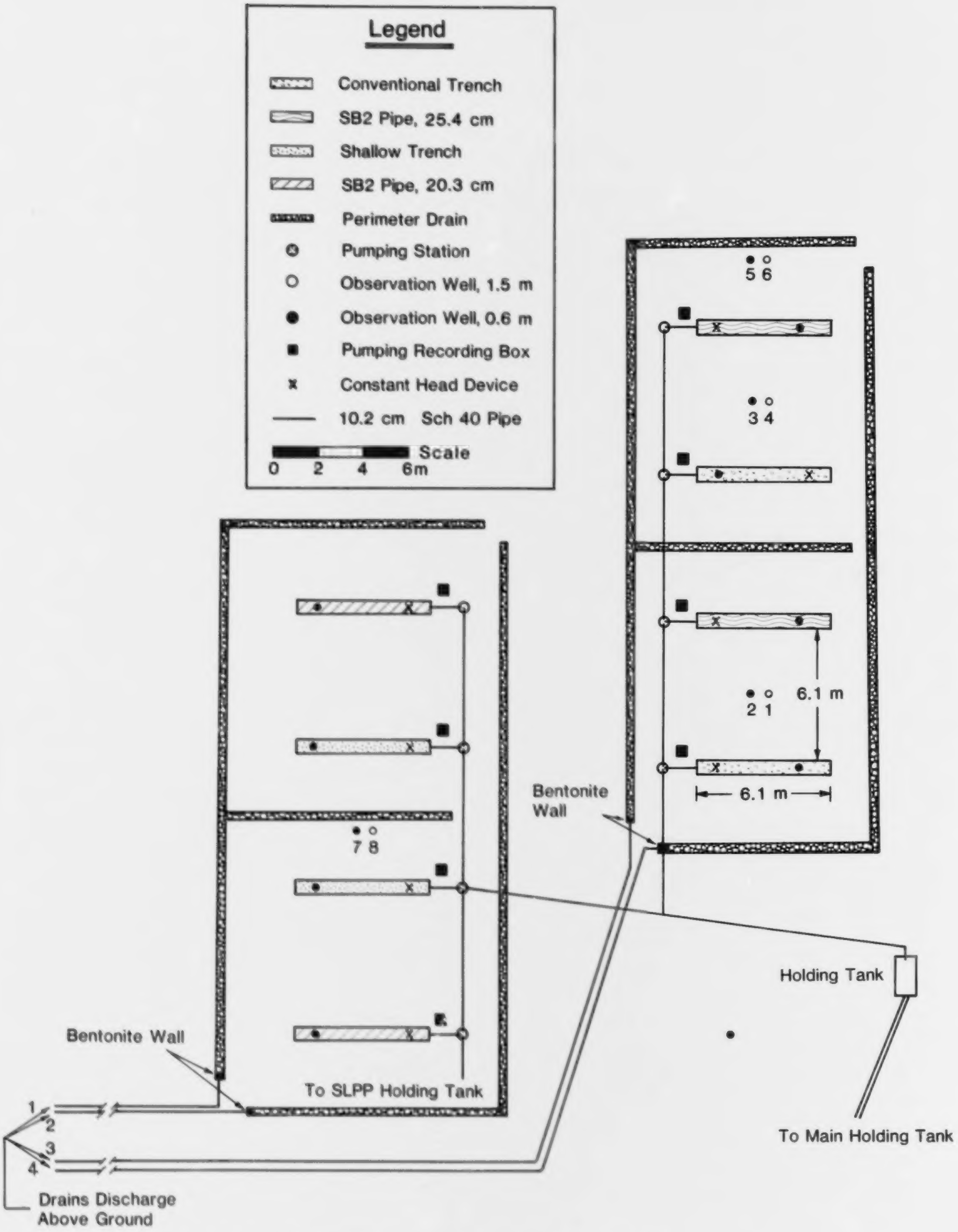

Legend
Conventional Trench
SB2 Pipe, 25.4 cm
Shallow Trench
SB2 Pipe, 20.3 cm
Perimeter Drain
Pumping Station
Observation Well, 1.5 m
Observation Well, 0.6 m
Pumping Recording Box
Constant Head Device
10.2 cm Sch 40 Pipe
Scale
0 2 4 6m
5 6
3 4
2 1
7 8
6.1 m
6.1 m
Bentonite Wall
Bentonite Wall
Holding Tank
To SLPP Holding Tank
To Main Holding Tank
1
2
3
4
Drains Discharge Above Ground

Figure 15. Location of trenches and perimeter drains.

based on a trench's capacity to absorb it.

In an attempt to lower the water table, perimeter drains were installed around the trenches during the second year of this study, as shown in Figure 15. The drains were located approximately 3 m from the trenches, and were 30 cm wide and 130 cm deep. The trenches in which the drains were installed were backfilled with gravel to a depth of 15 to 25 cm from the soil surface. The drains were divided into two sections in each area drained. One section intercepted groundwater upslope from the trenches, while the other collected water downslope and between the trenches. Each drain had a separate discharge pipe, which allowed for the collection of independent groundwater samples from each drain.

In addition to monitoring the wastewater absorption rates, septic tank effluent grab samples were collected once or twice per week throughout the study and analyzed for common wastewater constituents to determine the strength of the effluent.

The average concentrations of common wastewater constituents in the septic tank effluent are presented in Table 8. In general, the strength of the effluent was slightly weaker than concentrations reported in the literature (Otis & Boyle, 1976).

TABLE 8. CONCENTRATION OF SEPTIC TANK EFFLUENT APPLIED TO THE TRENCHES

Constituent	Mean	Range	Number of samples
	----------mg/l	------------	
Chloride	81.9	19.8-250.0	73
COD	181.0	26.0-358.0	47
NH_3-N	23.9	2.3-36.9	47
NO_2+NO_3-N	0.1*	<0.1-0.5	47
TKN	28.9	1.9-45.1	46
Total-P	11.8	0.6-17.9	46

* The mean was calculated by assuming a value of 0.1 mg/l for samples having a concentration less than the detectable limit of 0.1 mg/l.

Average monthly absorption rates for the three trench designs are shown if Figure 16. These values are the combined averages of the duplicate trenches. The average absorption rates varied greatly over the 2-year study. The shallow trench design consistently had a higher average absorption rate than the other two trench designs, while the conventional trench design had the lowest. There was also much variability from season to season. During the summer months when evapotranspiration and temperatures were high, and soil moisture contents low, all of the systems performed adequately. During the late fall, winter, and spring, however, absorption rates were much lower, and there were periods of time when no effluent was absorbed from any of the trenches. Absorption rates improved slightly during the second winter of this study, but none of the trench designs appeared to provide effective year-round solutions to the onsite wastewater disposal problems.

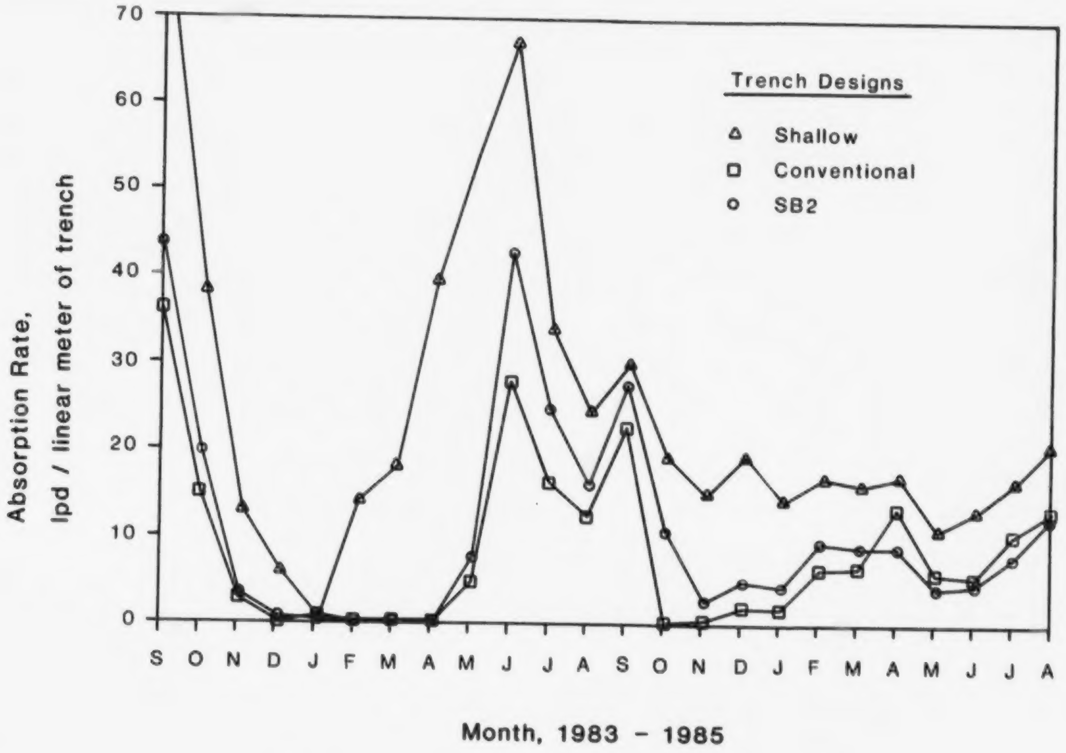


Figure 16. Monthly average absorption rates for each trench design.

The absorption rates of each trench design were grouped into 3-month intervals, based on seasons, and statistically analyzed using Analysis of Variance and Tukey's test (SAS Institute, Inc., 1985). These results are presented in Table 9. The shallow trench design had significantly higher absorption rates than the conventional or gravelless trench designs throughout the study, and from July to December 1984, the gravelless design had higher absorption rates than the conventional system. Overall, however, there was little difference in the absorption rates of the conventional and gravelless trench designs throughout most of the study. Anderson et al. (1983) also observed this similarity.

TABLE 9. EXPERIMENTAL TRENCH ABSORPTION RATES DURING MULTI-MONTH TIME INTERVALS (LPD PER LINEAR METER OF TRENCH)

	Trench design		
Time period	Conventional	25.4 cm SB2®	Shallow
Sep 1983	36* (A)†	43 (A)	73 (B)
Oct - Dec 1983	7 (A)	9 (A)	20 (B)
Jan - Mar 1984	0 (A)	0 (A)	15 (B)
Apr - Jun 1984	10 (A)	16 (A)	52 (B)
Jul - Sep 1984	17 (A)	24 (B)	30 (C)
Oct - Dec 1984	1 (A)	3 (B)	17 (C)
Jan - Mar 1985	6 (A)	8 (A)	16 (B)
Apr - Jun 1985	7 (A)	6 (A)	13 (B)
Jul - Aug 1985	12 (A)	10 (A)	19 (B)

* Average value of two trenches of the same design.

† Trench averages with the same letter for the same time period are not significantly different from each other.

While none of the trench designs evaluated were capable of disposing of septic tank effluent every month during the first year of the study, the shallow trenches had significantly fewer days of seasonal failure.

The absorption rates of the trenches were compared during the winter stress periods for the 2 years of this study to determine whether the perimeter drains were effective in increasing the absorption capacity of the soil. These results are presented in Table 10. The conventional and gravelless trench designs showed no significant difference between these two time periods. The absorption rate for the shallow trench was significantly different during the two stress periods, but absorption rates were lower during the second year. It is likely that factors such as clogging mat formation contributed to this decrease.

TABLE 10. AVERAGE ABSORPTION RATES DURING TWO WINTER/SPRING STRESS PERIODS (LPD PER LINEAR METER OF TRENCH)

Trench design	Stress period Oct 83-May 84	Stress period Oct 84-May 85*
Conventional	3† (A)§	5 (A)
Shallow	25 (A)	16 (B)
SB2®, 25.4 cm	4 (A)	6 (A)

* Perimeter drains were installed around the trenches in mid-October 1984.

† The average value for duplicate trenches of the same design is reported.

§ Trench averages with the same letter for the same trench design are not significantly different during the two stress periods.

During the 2 years of this study, there was no apparent increase in the absorption rate of the soil as the result of the installation of the perimeter drains. Although the groundwater table was slightly lower in the area of the trenches in this study during the winter after the drains were installed, it was also lower in areas not influenced by wastewater addition. This suggests that the second winter was drier, and an overall lower groundwater table existed.

Even if perimeter drains were effective in lowering the water table around soil absorption systems, the characteristics of the soils in this study suggest that only the macropores would be drained, while the micropores would remain filled with water. This would result in only a small increase in the percent of pore space available for absorption. Also, since water moves mainly through the macropores, it is possible that untreated effluent would channel through the macropores to the drains, eventually contaminating surface water supplies. More information on drains is needed in this area to determine whether drains are really effective in increasing absorption rates.

At the end of this study, the trenches were taken out of service and effluent was not applied to them for 2 weeks. After this 2-week resting period, the trenches were excavated to determine the extent of biological mat formation. Although the trenches had been continuously inundated with septic tank effluent for more than 2 years, the gravel above the distribution pipe in the conventional and shallow trenches was not covered with a gelatinous black slime. There was a thin black slime layer on the gravel located beneath the pipe, but this oxidized soon after the trenches were excavated. There was no indication of the presence of a biological mat at the gravel/soil interface, but slime and other deposits were observed inside the distribution pipe.

The gravelless trenches showed signs that septic tank effluent had been applied to them over an extended period. Effluent was present in the bottom of the corrugated tubing, and there was a noticeable buildup of gelatinous black slime. A well-developed black biological mat had also formed in the interface between the fabric and the soil, but the biomat extended only a few millimeters into the soil. There was no biological mat at the interface of the corrugations of the tubing and the fabric where the fabric pressed against the tubing, giving the fabric a striped appearance. As reported by Anderson et al. (1983), the biomat was noticeably thicker at the bottom of the corrugated tubing than on the sides or top.

The results of this study showed that all three trench designs can be used in the Avonburg soils at the study site to dispose of septic tank effluent during the summer months. During the winter and spring seasons, however, the conventional and gravelless trenches absorbed little or no effluent, and they were in a state of seasonal failure for months at a time. The shallow trenches had significantly fewer days of seasonal failure.

A comparison of the conventional gravel-filled trench and the SB2® corrugated fabric covered tubing showed that during 75% of the 2-year study, there was no statistically significant difference in the absorption rate of the conventional trenches and the gravelless trenches.

MOUND SYSTEM

The mound system is an alternative soil absorption system that relies on suitable sand-fill material that is mounded above the natural soil surface for treatment of wastewater. The purpose of the design is to overcome site restrictions such as slowly permeable soils, shallow soils over creviced or porous bedrock, and permeable soils with high water tables that prohibit the use of conventional soil absorption systems (USEPA, 1978). The absorption trenches or beds are located within the elevated sand mound, and effluent is pumped into them in prescribed doses. Treatment occurs in the fill material in the mound, and, in slowly permeable soils, the mound serves to improve absorption of the effluent by utilizing the more permeable topsoil. A more thorough description of the mound system is provided by USEPA (1980).

The dosing frequency for a mound system is typically set at four equal volume doses per day for adequate treatment and absorption (USEPA, 1980). The use of four doses per day has been demonstrated by several studies to provide adequate treatment of septic tank effluent (Boyle & Otis, 1982; Heidman & Bowker, 1981; Magdoff et al., 1974a).

In this study, dosing frequencies in simulated laboratory mound soil absorption systems were varied from the typical dose frequency to determine whether treatment of septic tank effluent was affected by the different dose rates. A summary of this study is presented here. A complete report is presented in Appendix E. Three 15-cm diameter sand/soil columns were constructed as shown in Figure 17 to simulate the mound systems. One column was dosed at 3-hr intervals, one at 6-hr intervals, and one at 12-hr intervals. The columns were dosed at a rate of 10 lpd/sq m. Sample collection began after the columns had been in operation for 6 months. Septic tank effluent and sand/soil column effluents were collected twice a week for approximately 3 months, and analyzed for biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), fecal coliforms, total phosphorus (Total-P), orthophosphate (Ortho-P), and nitrogen species.

The average concentrations of septic tank effluent (column influent) and effluent from the three columns for the parameters measured are presented in Table 11, along with the percent reduction of the various parameters through the columns. The average total nitrogen (Total-N) and COD concentrations of the septic tank effluent in this study were much higher than concentrations reported by other researchers, while the other parameters were within the range of other reported concentrations, although BOD_5 was near the higher range (Ronayne et al., 1982; USEPA, 1978). The reason for the high Total-N and COD concentrations is unknown.

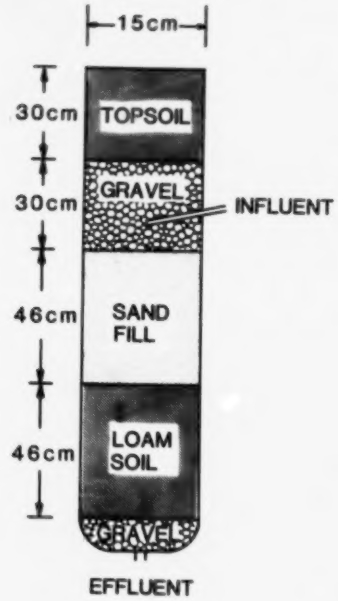


Figure 17. Cross section of the sand/soil column used to simulate a mound system.

Nearly complete removal of phosphorus, BOD_5, COD, fecal coliforms, and TSS occurred through all of the columns. Fecal coliform and BOD_5 concentrations were usually below the detectable limits of the tests in the column effluents. The column dosed every 3 hr produced an effluent with significantly lower total Kjeldahl nitrogen (TKN), ammonia (NH_3-N), and nitrate (NO_3-N) than those dosed every 6 and 12 hr. Magdoff et al. (1974a) also found nearly complete removal of fecal coliforms and COD in mound-simulated columns.

Total nitrogen concentrations were reduced by 52, 36, and 21% through the columns dosed on 3-, 6-, and 12-hr intervals, respectively. It is possible that biological denitrification resulted in this nitrogen removal. Denitrification has been shown to occur in an anaerobic zone that develops in the soil below the sand/soil interface as the result of decreased permeability at the interface (Magdoff et al., 1974a; Sikora & Corey, 1976). The sand fill is predominantly aerobic, while the underlying soil is anaerobic. This creates the potential for nitrification to occur in the fill and denitrification in the soil. Harkin et al. (1979) observed a loss of 22% of the Total-N and Magdoff et al. (1974b) calculated a loss of 32% of the Total-N in uncrusted columns dosed four times per day.

TABLE 11. MEAN VALUES FOR COLUMN INFLUENT AND EFFLUENT

Parameter	Septic tank effluent	3-hr eff.	6-hr eff.	12-hr eff.
Total-P	19.5[*]	0.3 (98)[†]	0.2 (99)	0.2 (99)
Ortho-P	20.6	0.1 (99)	0.1 (99)	0.1 (99)
BOD_5	306	<20[§] (93)	<20[§] (93)	<20[§] (93)
COD	578	57 (90)	52 (91)	59 (90)
Fecal coliform	$5.7x10^5$	<2 (99)	<2 (99)	<2 (99)
TSS	144	0[§] (99)	0[§] (99)	0[§] (99)
TKN	150	7.6 (95)	12.1 (92)	22.5 (85)
NH_3-N	124	7.2 (94)	11.9 (90)	22.5 (82)
NO_2-N	0.2	0.2 (0)	0.2 (0)	0.2 (0)
NO_3-N	0.3	64 (I)[**]	84 (I)[**]	96 (I)[**]
Total-N	150	72 (52)	96 (36)	119 (21)
pH[††]	6.5-8.1	7.3-8.1	7.4-8.0	7.5-8.1

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[*] Values are reported as mg/l, except fecal coliforms, which are reported as the number per 100 ml.

[†] Value in parentheses indicates percent reduction.

[§] Values were usually less than the detectable limit, therefore means are estimates.

[**] An increase occurred.

[††] Range of pH.

The variation in the total nitrogen removed in the columns in this study may have resulted from different flow regimes produced by the various dosing frequencies. In the 3-hr column, the small frequent doses would have created an unsaturated flow regime in the sand, which would maximize nitrification. The frequent application of effluent could have resulted in an essentially constant zone of saturation at the sand/soil interface, providing optimum conditions for denitrification. In the 12-hr column, sufficient effluent may have been applied to produce saturated flow in the sand. This would minimize nitrification, which in turn decreases the removal of nitrogen through denitrification since nitrate is essential for biological denitrification. Also, the infrequent application of effluent in the 12-hr column would provide more time for drainage to occur, reducing the depth of the saturated zone, thereby reducing denitrification.

Statistical comparisons among the laboratory sand/soil column effluents concentrations ($\alpha = 0.05$) determined that in this study, the dosing frequency did not affect the treatment of septic tank effluent with respect to the parameters measured, with the exception of nitrogen removal, which did show a significant difference. Data from larger-scale mound systems are needed before design recommendations can be made concerning the optimum dosing frequency, however.

GROUNDWATER CONTAMINATION FROM SUBSURFACE SOIL ABSORPTION SYSTEMS

A number of wastewater constituents have been found in groundwater as a result of contamination from soil absorption fields (Cogger & Carlile, 1984; DeWalle & Schaff, 1980; Reneau, 1977, 1979; Uebler, 1984). Between 1946 and 1960, 61% of all waterborne disease outbreaks in the U.S. were attributed to contaminated groundwater (Gerba et al., 1975). Groundwater contamination concerns become even greater in soils with seasonally high water tables, such as those in this study, because effluent may not be adequately treated before it enters the groundwater (Cogger & Carlile, 1984).

The groundwater near the LPP and the experimental trench design systems previously discussed was monitored in this study to determine whether contamination was occurring as the result of the added effluent. In the LPP system, the groundwater from both shallow (S) and deep (D) observation wells was sampled periodically over a 1-year period. The sampling procedure included pumping the wells dry prior to a dosing event using a small portable peristaltic pump and sterile plastic tubing. Following the dosing event the wells were left to recharge for a period of 3 to 4 hr and then groundwater samples were collected from each well. These samples were analyzed for contaminants that are typically monitored in groundwater, including nitrogen species, phosphorus, fecal coliforms, and chloride.

The concentrations of the measured parameters from the LPP system are given in Table 12. Of these parameters, chloride undergoes no biological transformations, nor is it adsorbed onto soil particles, and therefore is a useful tracer or indicator of septic tank effluent movement through soil (Canter & Knox, 1984).

The chloride concentrations obtained from observation wells located 1.5 m downslope from the system (Figure 13, Wells 3S, 3D) were significantly higher than concentrations obtained from control wells (Figure 13, Wells 1S, 1D). This suggests that septic tank effluent moved laterally from the trenches in a downslope direction and intercepted the groundwater. Others have reported the movement of chloride from subsurface soil absorption fields (Reneau, 1977, 1979; Viraraghavan & Warnock, 1976).

The chloride concentration of the groundwater in this study steadily increased over time. Brown and Slowey (1977) found that the concentration of chloride in the leachate from soils with clay, sandy clay, and sandy loam textures at several study sites increased throughout a 2-year study period.

Although the chloride concentrations suggest that effluent was moving into the groundwater, none of the other parameters tested showed an increased concentration in the wells located downslope from the system.

The nitrogen concentrations for all of the tested species were low compared with values reported in the literature for groundwater collected near soil absorption fields (Cogger & Carlile, 1984; Reneau, 1977, 1979; Uebler, 1984), and NO_3^--N concentrations were greatly below the U.S. EPA Drinking Water Standards limitation of 10 mg/l NO_3^--N in drinking water supplies (USEPA, 1974). However, other researchers have reported large variability in the NH_3^+-N and NO_3^--N concentrations as the result of time and soil moisture content (Brown et al., 1984; Reneau, 1979). Therefore, because the groundwater in this study was only sampled for 1 year, it is possible that ammonium was still being adsorbed onto exchange sites in the soil beneath the absorption trenches, and that higher nitrogen concentrations could occur over time.

There was also no significant difference in total phosphorus (Total-P) concentrations between the groundwater from the control and the downslope wells in this study. Others have also reported that the presence of an operating soil absorption field had little impact on the phosphorus concentration of groundwater obtained from observation wells located near the field (Brown et al., 1977; Canter & Knox, 1984; Carlile et al., 1981). However, other studies have shown that phorphorus movement from drainfields occurred (Cogger et al., 1983; Reneau, 1979; Sawhney & Starr, 1977; Viraraghavan & Warnock, 1976). This movement has been shown to be affected by the length of period of groundwater saturation and system age (Cogger et al., 1983; Reneau, 1979). In most cases, the concentration of phosphorus in the groundwater decreased rapidly with distance.

Although phosphorus from the trenches did not appear to be entering the groundwater during the time span of this study, the average total phosphorus concentrations in both the control and the downslope wells exceeded the typical phosphorus concentrations of 0.01 to 0.1 mg/l in agricultural soil solutions (Brown et al., 1977). The reason for the higher values is not known.

Results from a limited number of samples tested for fecal coliform organisms showed that no significant difference occurred between the groundwater in the control and the downslope wells in this study, and that numbers of organisms were relatively low. Sixty-three percent of the samples from all of the wells had less than the detectable limit of 2 fecal coliforms per 100 ml.

Others have found that groundwaters can be contaminated with fecal organisms from soil absorption systems (Reneau, 1978; Viraraghavan & Warnock, 1976). In most of these studies, quantities of fecal organisms diminished rapidly with distance, and the extent of fecal coliform contamination was greater in permeable soils and in groundwater surrounding wetter systems.

There was no significant difference in the concentrations of the measured parameters between the groundwater from shallow and deep groundwater monitoring wells.

TABLE 12. GROUNDWATER CONCENTRATIONS FROM OBSERVATION WELLS LOCATED NEAR THE LPP SOIL ABSORPTION FIELD

Parameter	Location	Number of samples	Mean	Median	Range
Chloride mg/l	Well 1S	25 (0)*	7.3 (A)†	2.7	0.9-27.3
	Well 1D	26 (1)	7.2 (A)	3.2	<0.1-36.9
	Well 3S	26 (1)	36.4 (B)	23.9	8.4-140.0
	Well 3D	32 (2)	35.9 (B)	29.3	<0.1-99.5
NH_3-N mg/l	Well 1S	26 (18)	0.2 (A)	0.1	<0.1-0.8
	Well 1D	31 (14)	0.4 (A)	0.1	<0.1-3.0
	Well 3S	26 (19)	0.2 (A)	0.1	<0.1-0.8
	Well 3D	26 (17)	0.1 (A)	0.1	<0.1-0.6
NO_2 + NO_3-N mg/l	Well 1S	26 (16)	0.4 (A)	0.1	<0.1-6.1
	Well 1D	31 (13)	0.6 (A)	0.1	<0.1-6.1
	Well 3S	26 (20)	0.1 (A)	0.1	<0.1-0.3
	Well 3D	26 (16)	0.1 (A)	0.1	<0.1-0.3
TKN mg/l	Well 1S	26 (4)	1.0 (A)	0.9	<0.1-2.7
	Well 1D	31 (5)	1.1 (A)	0.9	<0.1-8.2
	Well 3S	26 (5)	1.0 (A)	1.1	<0.1-2.8
	Well 3D	26 (6)	0.7 (A)	0.4	<0.1-2.3
Total-P mg/l	Well 1S	26 (7)	0.9 (A)	0.9	<0.1-3.2
	Well 1D	31 (7)	0.8 (A)	0.5	<0.1-6.5
	Well 3S	26 (5)	1.3 (A)	1.1	<0.1-5.9
	Well 3D	25 (5)	1.2 (A)	1.1	<0.1-4.3
Fecal coliforms #/100 ml	Well 1S	8 (4)§	15 (A)	2	<2-79
	Well 1D	8 (5)	<2 (A)	<2	<2-8
	Well 3S	9 (6)	20 (A)	<2	<2-130
	Well 3D	8 (6)	<2 (A)	<2	<2-8

* Number of samples having a concentration less than the detectable limit of 0.1 mg/l. Means and medians were calculated by assuming a value of 0.1 mg/l for these samples.

† Means with the same letter for the same parameter are not significantly different at the 95% confidence level as determined by Duncan's Multiple Range Test.

§ Number of samples having less than 2 fecal coliforms/100 ml (MPN method).

The groundwater surrounding the experimental trench design system was also monitored on a limited basis for typical wastewater parameters. In this system, grab samples were collected from the surface discharging perimeter drains during the spring of 1985. The locations of the drains are shown in Figure 15. The samples were analyzed for chloride, NH_3^+-N, NO_2^- + NO_3^--N, TKN, total phosphorus (Total-P), and fecal coliforms. The results of these analyses are shown in Table 13.

TABLE 13. CHARACTERISTICS OF GROUNDWATER OBTAINED FROM THE DISCHARGING PERIMETER DRAINS

Parameter	Background well*	Drain 1	Drain 2	Drain 3	Drain 4
Chloride	7.2†	26.8	30.8	11.1	7.8
NH_3-N	0.4	0.4	0.5	0.1	0.2
NO_2+NO_3-N	0.6	1.9	2.7	1.9	1.8
TKN	1.1	3.1	2.1	1.8	1.5
Total-P	0.8	0.8	0.5	0.6	0.6
Fecal coliforms	<2	41	12	14	32
Max, recorded flow, lpm	N/A	0.8	1.1	1.3	5.9

* 150 cm deep groundwater sampling well not affected by the trenches and used as a control (Effert & Cashell, 1985).

† Values are averages reported as mg/l, except fecal coliforms, which are reported as the average number per 100 ml.

The average chloride concentration in the groundwater from two of the drains was greater than that found in the groundwater in a well at a location not influenced by the trenches, indicating that contaminated groundwater intersected several of the perimeter drains. Nitrate + nitrite-nitrogen concentrations were also slightly higher, but still below the U.S. EPA Drinking Water Standards of 10 mg/l. The range of fecal coliforms never exceeded 200/100 ml.

In summary, it appeared that groundwater contamination from the subsurface soil absorption systems in this study was not a concern for the parameters measured and for the time that the systems were monitored. However, many researchers have shown that contaminant breakthrough did not occur for a long time period after the system was put into operation (Brown et al., 1977, 1984; Cogger et al., 1983; DeWalle & Schaff, 1980), or that it occurred only during certain times of the year (Reneau, 1978). Therefore, the groundwater around systems installed in soils with seasonally high groundwater tables such as those in this study should be continuously monitored for several years to determine the potential for groundwater contamination from wastewater.

CAPACITY OF SOILS TO ACCEPT WASTEWATER

The successful operation of a soil absorption field system not only requires adequate treatment of the wastewater before it enters surface water or groundwater, but also requires acceptable absorption and percolation of the volume of wastewater produced. If wastewater percolates too quickly through the soil profile, adequate treatment may not occur and groundwater contamination is possible. However, if the soil is slowly permeable and cannot absorb and transmit the wastewater produced, the wastewater will either back up or will break out on the soil surface. Therefore, an understanding of how water moves into and through soil is necessary to predict the potential of soil for wastewater absorption and treatment (USEPA, 1980).

The soil moisture regimes surrounding several of the wastewater absorption trenches in this study were monitored to determine the capacity of the soil to absorb and transmit wastewater. The soil absorption systems that were monitored included the LPP system, the alternating field system, the gravelless trenches, and the shallow trenches that were previously described.

The soil moisture regimes around the systems were monitored by several methods. In the alternating field system, commercially available 1.9-cm diameter and 5-cm long porous cup tensiometers were installed in both the existing and alternating systems at the locations shown in Figure 8. Tensiometers were installed in the bottom of the trenches and at various heights in the trench sidewalls, as shown in Figure 18.

In the shallow-installed trenches, commercially available 1.9-cm diameter and 5-cm long porous cup tensiometers were installed following the procedure of Bouma et al. (1974) at the locations indicated in Figure 19. Trench ponding depth and groundwater table elevation monitoring wells, constructed of 5-cm diameter plastic pipe, were also installed.

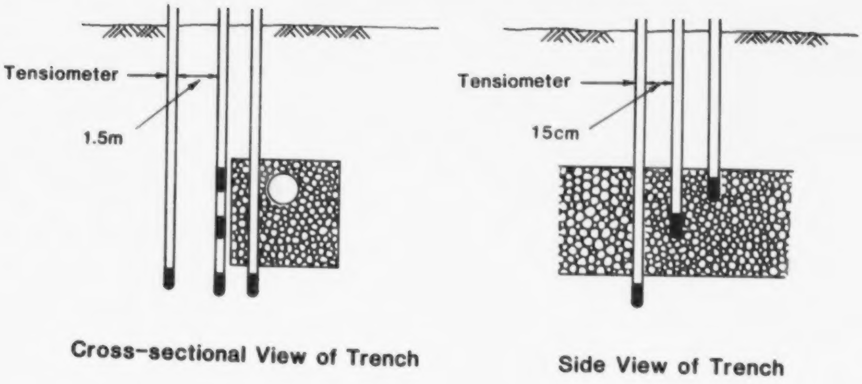

Tensiometer
1.5m
Cross-sectional View of Trench
Tensiometer
15cm
Side View of Trench

Figure 18. Location of tensiometers in alternating soil absorption field trenches.

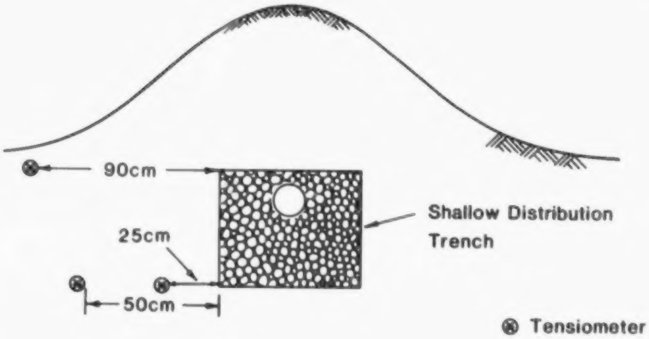

90cm
25cm
50cm
Shallow Distribution Trench
Tensiometer

Figure 19. Location of tensiometers in shallow trenches.

In the gravelless trenches, one tensiometer was installed in the natural soil in the trench bottom and another tensiometer was installed in the backfilled soil above the 25-cm diameter corrugated plastic pipe, as shown in Figure 20.

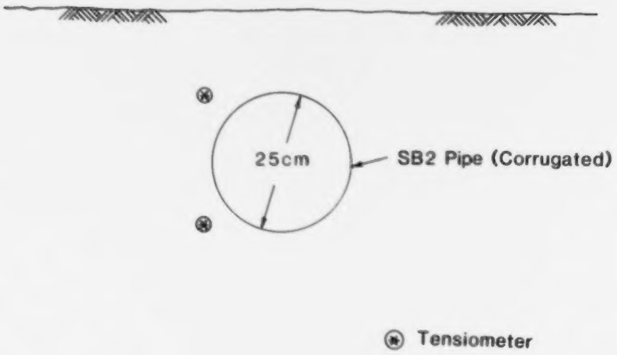


Figure 20. Location of tensiometers in gravelless trenches.

In the LPP system, pencil-sized tensiometers (Bouma et al., 1974) connected to mercury manometers were installed during construction in the trench bottoms to monitor the soil moisture potential below the trenches. Three additional tensiometers were installed midway between two of the trenches to monitor the soil moisture potential at a distance from the trenches. These commercially available 1.9-cm diameter and 5-cm long porous cup tensiometers were connected to mercury manometers, and were installed by the procedure of Bouma et al. (1974).

Soil moisture contents surrounding this system were measured with a depth moisture gauge (neutron probe) manufactured by Troxler Electronic Laboratories Inc. (Model 3222). Access tubes for the gauge were installed to a depth of 150 cm midway between the trenches, downslope from the system, and at several locations beyond the influence of the system. The groundwater table elevation was also monitored in these locations. The groundwater monitoring wells were constructed from 5-cm diameter plastic pipe that was perforated in the lower 60 cm, and were installed to a depth of approximately 150 cm. The ponding depth in the trenches was also monitored with the use of 5-cm diameter plastic pipe observation wells. The locations of both the groundwater and ponding depth monitoring wells are shown in Figure 13.

The average soil moisture potentials below the trench bottoms of the various trenches in this study are presented in Table 14, along with values reported by others. Positive soil moisture potentials were present below the infiltrative surface of the trench in all of the trenches in this study throughout the monitoring period, indicating that several centimeters of saturated soil were present below the trench. These results are contrary to what other researchers have found for soil absorption systems of similar age installed in soils with textures similar to those in this study. Unsaturated conditions have been shown to occur below soil absorption trenches as the result of clogging mat formation at the gravel/soil interface. Bouma (1975) found soil moisture potentials of -6 and -15 cm, indicating unsaturated conditions, in two systems installed in a clay-textured soil.

TABLE 14. AVERAGE SOIL MOISTURE POTENTIAL BELOW SOIL ABSORPTION TRENCHES

Trench type	Soil texture*	Age of system	Soil moisture potential (cm)	Reference
LPP	sicl	12 mo	+20	
Shallow	sicl	16 mo	+23	
	sicl	16 mo	+45	
Gravelless	sicl	12 mo	+24	
	sicl	12 mo	+14	
Other studies	c	12 mo	-6	Bouma (1975)
	c	24 mo	-15	Bouma (1975)
	c	4 mo	-6	Bouma (1971)

* sicl = silty clay loam; c = clay.

It is possible that saturated conditions were observed in this study because clogging mat formation had not proceeded to the extent of reducing flow into the soil, and that saturated flow was occurring for at least several centimeters below the system. Physical observation of the infiltrative surface of several of the trenches after 2 years of service indicated that very little clogging mat formation had occurred even though the trenches had been inundated nearly continuously.

It is also possible that saturated conditions were observed because of the manner in which the tensiometers were installed. The tensiometers in most of the trench bottoms were installed vertically into the soil below the trench, and it is possible that water channeled around the shaft to the porous cup, creating saturated conditions in the area surrounding the cup. However, the tensiometers were carefully installed with soil packed tightly around the shaft to avoid channeling. Also, in the gravelless trenches, the tensiometers were installed on an angle so that direct vertical channeling could not occur. Because positive potentials were seen in all of the trenches with tensiometers, it is likely that the tensiometer readings were valid.

Another study performed on these soils and presented in this report (Section 7) showed that the actual average soil absorption rate from a conventional trench during the summer months corresponded to a soil moisture potential of -3.5 cm. This suggests that nearly saturated flow was occurring from the trenches.

The soil moisture potentials in the sidewall of one of the alternating soil absorption field system trenches throughout the second summer of monitoring indicated that saturated conditions were also dominant in the sidewall at the same elevation as the bottom of the trench (Figure 21). Unsaturated conditions existed in the sidewall 15 and 30 cm above the trench bottom. These conditions were present even though the ponding depth in the trench was greater than 30 cm throughout this monitoring period. Because flow occurs more rapidly under saturated conditions (Hillel, 1980a), it is likely that most of the flow in this system occurred out of the bottom of the trench. However, the tensions observed in the sidewalls were low compared with the tensions in a control area during this time period, which were often above the practical measurable limit of a tensiometer (-800 cm, Richards, 1965), suggesting that some of the wastewater was removed from the trench through capillary flow from the sidewall areas.

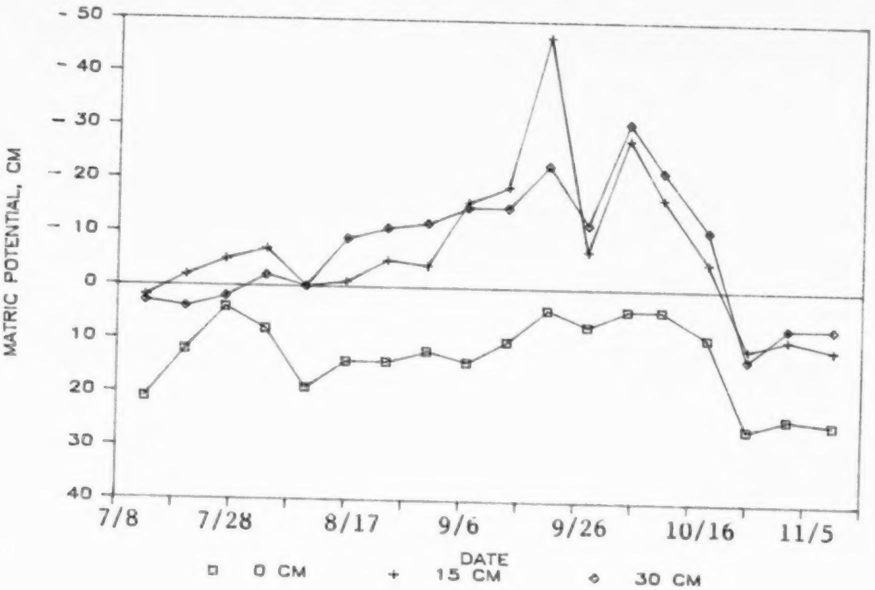


Figure 21. Soil moisture potentials in sidewalls of trenches in the alternating system. The depths indicate the height of the tensiometers above the bottom of the trench.

The soil moisture potentials a short distance away from one of the shallow trenches that had been in operation for 2 years showed saturated conditions at 25 and 50 cm from the trench at the same elevation as the bottom of the trench until July, when unsaturated conditions dominated (Figure 22). The soil 90 cm from the trench, but at an elevation equal to the top of the trench, was unsaturated throughout this monitoring period. Although unsaturated conditions prevailed a short distance from the trenches during the summer months, the tensions were low compared with tensions in control areas, indicating that the soil around the trenches remained relatively wet throughout the year. Even the soil 90 cm from the trench at a very shallow depth where most water is removed during the summer months by evapotranspiration remained relatively moist. These low soil moisture tensions indicate that the additional effluent from the soil absorption trenches affected the soil for at least 1 m surrounding the trench, and indicate that the soils have a high capillary potential to absorb moisture.

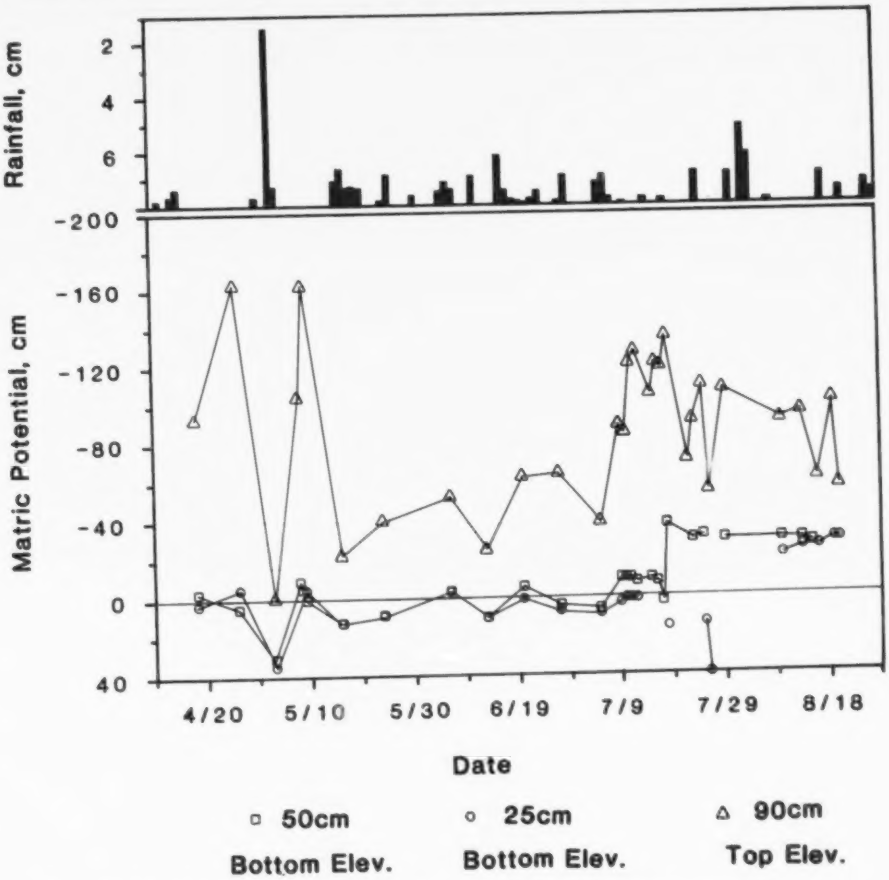


Figure 22. Soil matric potential in soil surrounding shallow trenches.

The influence of wastewater applied to these soils was also seen in the LPP system, where mostly saturated conditions were present midway between two of the trenches at depths of 23, 53, and 74 cm (Figure 23). From the end of June through the end of August, the soil at 53 and 74 cm was saturated. Nearly saturated conditions existed at 23 cm. The system was then taken out of service toward the end of August, at which time the soil at 23 cm became unsaturated, and soil matric potentials became greater than -800 cm, the practical upper limit of a tensiometer (Richards, 1965). The soil at 53 cm also became unsaturated, while at 74 cm, nearly saturated conditions continued.

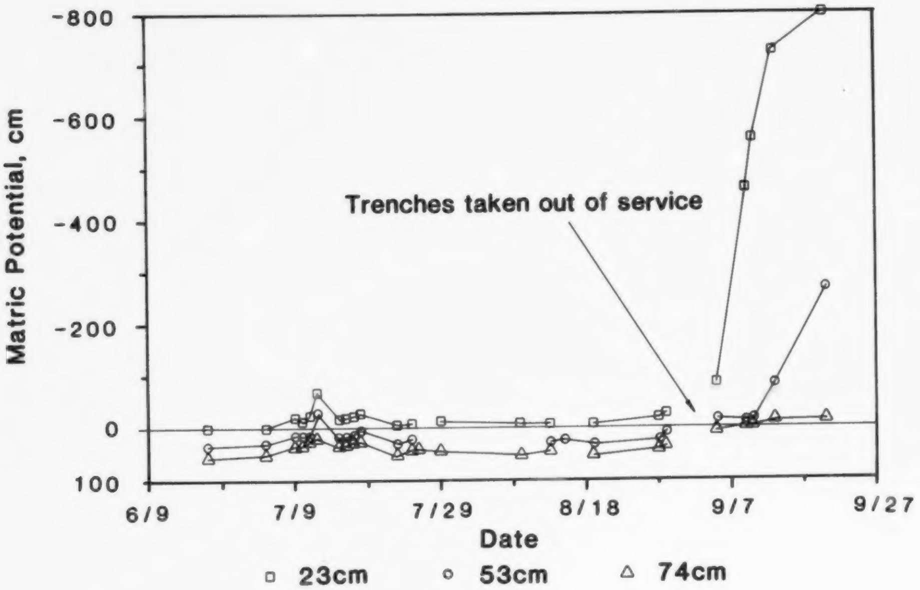


Figure 23. Soil matric potential between LPP trenches.

Figure 24 shows the variation in the soil moisture content with depth for two different dates between two trenches in the LPP system, downslope from the LPP system, and in an area not influenced by subsurface addition of effluent (control). The dates represent the nearly saturated conditions during the winter months (12/3/84) and the drier conditions during the summer months (7/20/85). During the summer months in the control profile, the soil moisture content decreased to a depth of at least 90 cm, with the greatest decrease in the surface horizon as the result of greater evapotranspiration and higher temperatures. At the deepest depth monitored, 120 cm, only a slight decrease in the soil moisture content was seen during the summer months, and the change was within the statistical error of the neutron probe.

The two soil moisture profiles between the LPP soil absorption trenches showed that no change occurred in the soil moisture content between winter and summer. At a distance of 5 m downslope from the system, the soil moisture content decreased during the summer months in the top 60 cm of the soil profile, but little change occurred below this depth, probably as the result of groundwater mounding below the system. The influence of the addition of effluent on the soil moisture content surrounding the trenches was great.

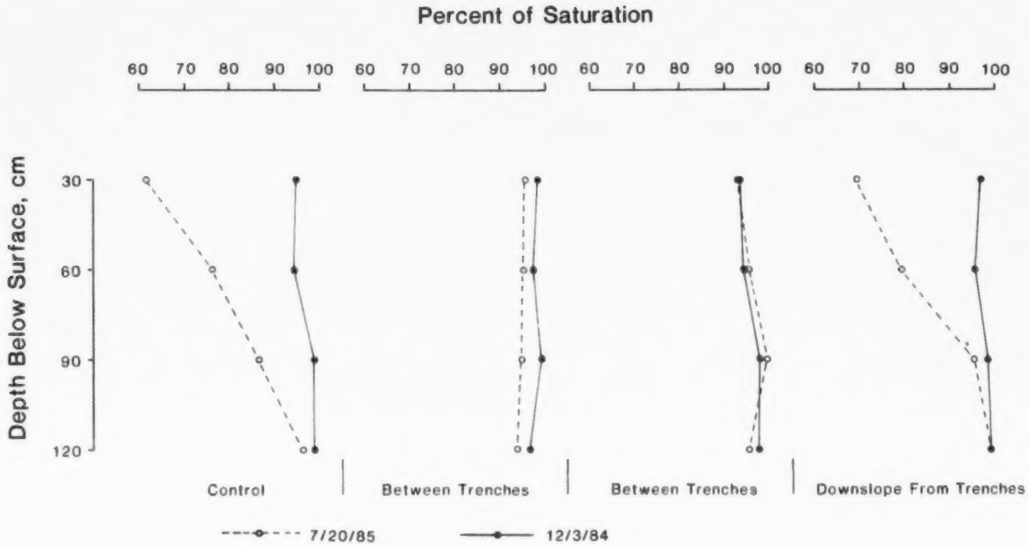


Figure 24. Soil moisture content change with depth over two time periods and four locations around LPP system.

All of these examples (Figures 21, 22, 23, and 24) show that the soil moisture regime surrounding the absorption trenches remained nearly constant throughout the monitoring periods. The soil was slightly drier during the summer months, but the change that occurred was minimal compared with areas that were not influenced by the addition of effluent, such as that seen in Figure 7 (Section 4). This suggests that water was absorbed from the systems in a relatively steady state condition.

The addition of effluent to the soil also resulted in groundwater mounding below the systems. Figure 25 gives the depth to the groundwater table within and immediately surrounding the LPP system, the background groundwater table elevation in an area not influenced by the addition of effluent, and the ponding depth in the trenches prior to a dosing event for one time period. On this date, the baseline groundwater table was below approximately 1.5 m, the detection limit of the well. Below the LPP system, however, the water table was closer to the soil surface. Approximately 1.5 m upslope and downslope from the system the groundwater table was approximately 60 cm from the soil surface, and within the system the groundwater table was approximately 30 cm from the soil surface. All but the most upslope trench had ponded effluent prior to the dosing event.

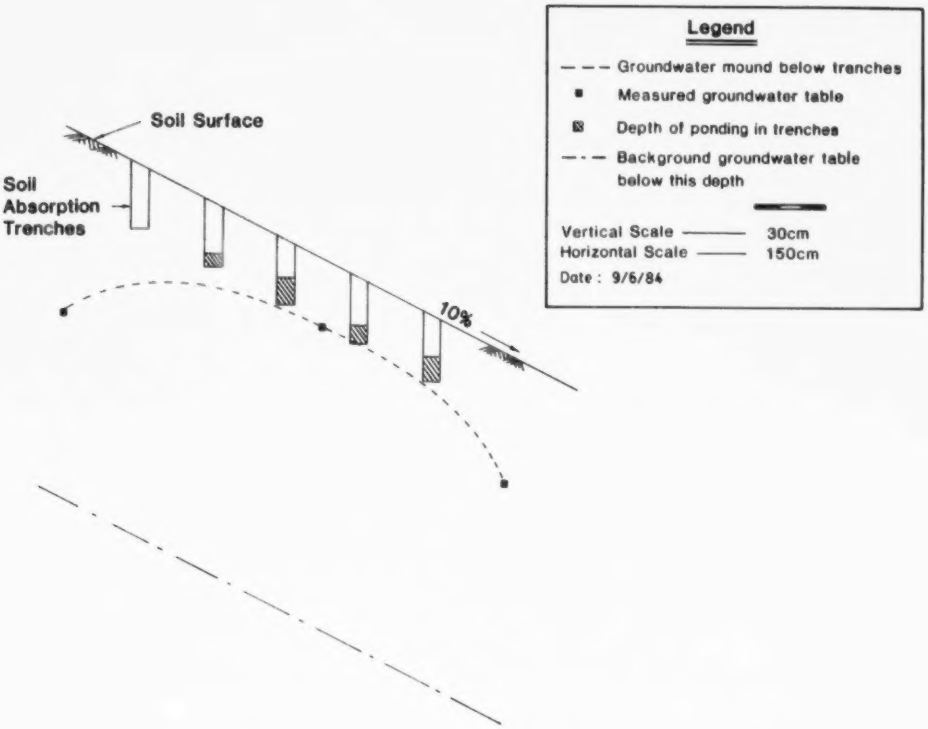

Soil Surface
Soil Absorption Trenches
10%
Legend
Groundwater mound below trenches
Measured groundwater table
Depth of ponding in trenches
Background groundwater table below this depth
Vertical Scale 30cm
Horizontal Scale 150cm
Date : 9/6/84

Figure 25. Groundwater mounding below LPP system.

Groundwater was also continuously mounded in the alternative trench design area. The average groundwater table depth in this area was less than 50 cm below the soil surface from October to May. During the summer months, the groundwater table was approximately 75 cm below the soil surface in this area, while it was greater than 1.5 m in areas not influenced by the addition of effluent.

The groundwater mounding observed below the LPP system most likely resulted from the additional moisture contributed from the effluent. As previously mentioned, the soils in this study were nearly saturated below approximately 1 m throughout the year, even when a water table was not present. Therefore, only a small volume of water would be required to fill the remaining air-filled pores and develop a free water surface.

The soil moisture potential, soil moisture content, and groundwater table data obtained in this study suggest that the soils investigated were able to absorb a large volume of water during the summer months as the result of the high capillary potential of the micropores and the low soil moisture contents in the upper soil horizons. Because the systems in this study that were installed at shallower depths utilize more of the upper soil horizons, this helps to explain why they performed better than the systems installed at deeper depths. The absorption capacity greatly decreased during the winter months because of the high soil moisture content and high groundwater table.

Although the soils had a high capacity to absorb wastewater when storage capacity was available, transmission of the water was slow throughout the year, and especially during the winter months. During the summer months, transmission occurs mostly through unsaturated flow. A hydraulic gradient, the driving force acting on a liquid (Hillel, 1980a), is induced by evapotranspiration, which can remove large volumes of water (Hillel, 1980b), and by capillary and gravitational potentials. Water that moves downward eventually reaches the groundwater mounded below the system. This water can then spread out by capillary action, or can move laterally in a thin saturated zone over the relatively nonporous bedrock. In winter, however, evapotranspiration is negligible, and because of the high soil moisture content, the capillary potential of the soil becomes less. Because the groundwater table below the system is very close to the infiltrative surface during the winter, most of the wastewater probably enters the groundwater system where it moves mainly by gravitational flow. However, because a groundwater table exists over the entire site and not just under the absorption trench, the gradient for flow is low, and groundwater movement occurs slowly. Therefore, more water can be absorbed and removed from a soil absorption trench in these soils during the summer months.

DISCUSSION OF SOIL ABSORPTION SYSTEMS

Of the systems that were studied that rely on the soil for treatment and disposal of wastewater, only the low-pressure pipe system and the shallow trench system appeared to perform satisfactorily during most of the year, with only occasional seasonal failure. The alternating soil absorption field system also appeared to function adequately during most of the year, but because it was installed in fill material and is not typical of conditions found southwestern Ohio, its performance was not compared with the other systems in this study.

A comparison of the loading rates for the LPP and the shallow systems is presented in Table 15. The loading rates in Table 15 were calculated based on the area of the trench bottom. This is different than the way in which the loading rates were previously presented. In the LPP system, the loading rate, as shown in Table 7, was based on the entire area that the system covers, as is typically reported. The loading rates for the shallow system, as shown in Table 9, were based on a linear meter of trench.

Since the LPP and the shallow systems were installed at the same depth, the main difference in comparing the loading rates based on bottom trench area is that absorption of effluent through the trench sidewall is not taken into account. In the shallow trench, effluent was always ponded at a relatively constant depth and it is likely that some absorption occurred through the sidewall. In the LPP system, however, effluent was dosed into the trenches, and they were either dry or had only a small ponding depth between doses. Therefore, not as much sidewall absorption would have occurred. However, as previously discussed in this section, most of the flow occurred out of the trench bottom in the systems in this study. Comparing the loading rates based only on the bottom area provides a relative comparison between the performance of the two systems.

TABLE 15. A COMPARISON BETWEEN THE LOADING RATES OF THE LPP AND SHALLOW TRENCH SYSTEMS

Date	Loading rate (lpd/sq m) LPP system	 Shallow system
Sep 1983 - Apr 1984	24	29
Apr 1984 - Jun 1984	41	87
Jul 1984 - Sep 1984	54	50
Oct 1984 - Dec 1985	24	27

The LPP and the shallow trench systems had similar absorption rates throughout most of the study period, except between April and June 1984, when the shallow trench system had a much higher loading rate. For the last 1.5 yr of the study, the loading rates of the two systems were similar.

Although the average loading rates of these systems were similar, the shallow system had more instances of surface seepage. This was probably in part the result of slope and the depth to the high groundwater table. The shallow system was installed in an Avonburg soil on a 0 to 2% slope. Little runoff or lateral flow would occur on this landscape position. The LPP system was installed on a 10% slope in a Rossmoyne soil, which differs from the Avonburg soil only in that it has a lower seasonally high groundwater table. More runoff and lateral flow would occur on the 10% slope.

Although both of these systems appeared to perform adequately during most of the year, both of them experienced occasional seasonal failure, which is unacceptable for health and nuisance reasons. Therefore, before they are recommended for use, further studies should be performed to determine if design modifications could eliminate seasonal failure. Recommended design modifications include increasing the trench spacing to provide as large of a lateral absorption area as possible, since soil moisture potential and soil moisture content data indicate that the soils have a large capillary potential to absorb wastewater. The potential for freezing in the shallow systems should be studied further, and the need for mounding soil above the shallow trench system to prevent freezing should also be investigated since this adds considerable expense to the installation of the system, and can make the system aesthetically unacceptable.

Because of the seasonal high water table in these soils, the possibility of groundwater contamination should be further evaluated.

Because of the lack of centralized sewer systems and suitable soils in many areas of southwestern Ohio, the LPP and shallow trench systems are possible alternatives for wastewater disposal in areas where soil absorption is the only option available. Areas should be evaluated on a site-by-site basis to determine if occasional seasonal failure is an acceptable risk.

SECTION 6

ALTERNATIVE SYSTEMS THAT DO NOT RELY ON THE SOIL FOR DISPOSAL OF WASTEWATER

INTRODUCTION

Alternative onsite wastewater treatment and disposal systems can overcome many limitations for subsurface disposal. However, some soils and sites are so totally unsuitable as treatment and disposal media that they require onsite wastewater treatment systems that are not dependent upon soil disposal. Such alternative systems are designed to discharge the treated wastewater to surface waters or to the atmosphere.

Alternative systems that rely on the soil for disposal have been shown to be only partially effective in the soil and site conditions present in southwestern Ohio (Section 5). Unless very low loading rates are used, even systems such as the Low Pressure Pipe system will not function properly during those times of the year when the water table is high.

Various alternative systems that do not rely on the soil for disposal were evaluated for use in southwestern Ohio. The systems studied that discharge to surface waters included intermittent sand filters, recirculating sand filters, upflow gravel filters, and subsurface discharging gravel beds. Systems that discharge to the atmosphere, evapotranspiration systems, were also evaluated.

A summary of these studies is presented in this chapter.

SYSTEMS THAT DISCHARGE TO SURFACE WATERS

Direct discharge of onsite treatment system effluent is a disposal option if an appropriate receiving water is available and if the regulatory agencies permit such a discharge. The level of treatment required varies, depending on local regulations, stream water quality requirements, and other site-specific conditions (USEPA, 1980).

Septic tanks and aerobic units, the two most typical pretreatment units for onsite wastewater treatment and disposal, do not treat the effluent sufficiently for direct disposal to surface waters (USEPA, 1978, 1980). Some type of polishing unit such as a sand or gravel filter is required to further treat the effluent in order to render it acceptable for surface discharge. In addition, disinfection of effluent to destroy pathogenic organisms in the wastewater stream is usually required if surface discharge is used.

Various types of polishing units, including full-scale field units and scaled-down laboratory units, were investigated in this study. These included:

- Full-scale intermittent sand filters
- Laboratory-scale intermittent sand filter columns
- Laboratory-scale recirculating sand filter columns
- Full-scale upflow gravel filters
- Full-scale discharging subsurface gravel beds

Intermittent Sand Filters

Intermittent sand filtration is based on the intermittent application of wastewater to a bed of granular material that is underdrained to collect and discharge the final effluent. This process will produce effluents of very high quality if the filters are properly designed, operated, and constructed (USEPA, 1980).

Intermittent sand filters are normally used to polish effluents from septic tanks or aerobic units. The wastewater from these pretreatment systems is collected in a pumping chamber and then pumped to the top of the sand filter at a variable dosing frequency.

Figures 26 and 27 are cross-sectional and plan views, respectively, of a typical intermittent sand filter. The filters are usually constructed of precast concrete. A perforated PVC pipe is placed in the bottom of the box in about 20 to 30 cm of washed gravel to collect the treated effluent after it has passed through the filter. A layer of washed pea gravel 7 to 12 cm thick is usually placed above the layer of washed gravel to prevent the sand from filtering into the lower gravel layer and underdrain. The top layer of the filter is typically 76 cm of washed sand, the primary filtration material. A wastewater distribution system applies the wastewater to the sand surface as uniformly as possible. In most cases these filters are covered with some type of wooden cover to keep animals out of the filter and to reduce the possibility of accidental contamination. This wooden cover is removable to allow for filter maintenance.

Although sand filtration is one of the oldest methods of wastewater treatment known (USEPA, 1980), specific design criteria for the filters have not been adequately researched and defined. Also, not much research has been performed to determine the extent of pretreatment necessary to produce a high quality effluent, and only a small amount of information is available concerning the operation and maintenance requirements of full-sized onsite intermittent sand filters. These issues were addressed in both full-scale and laboratory models in this study.

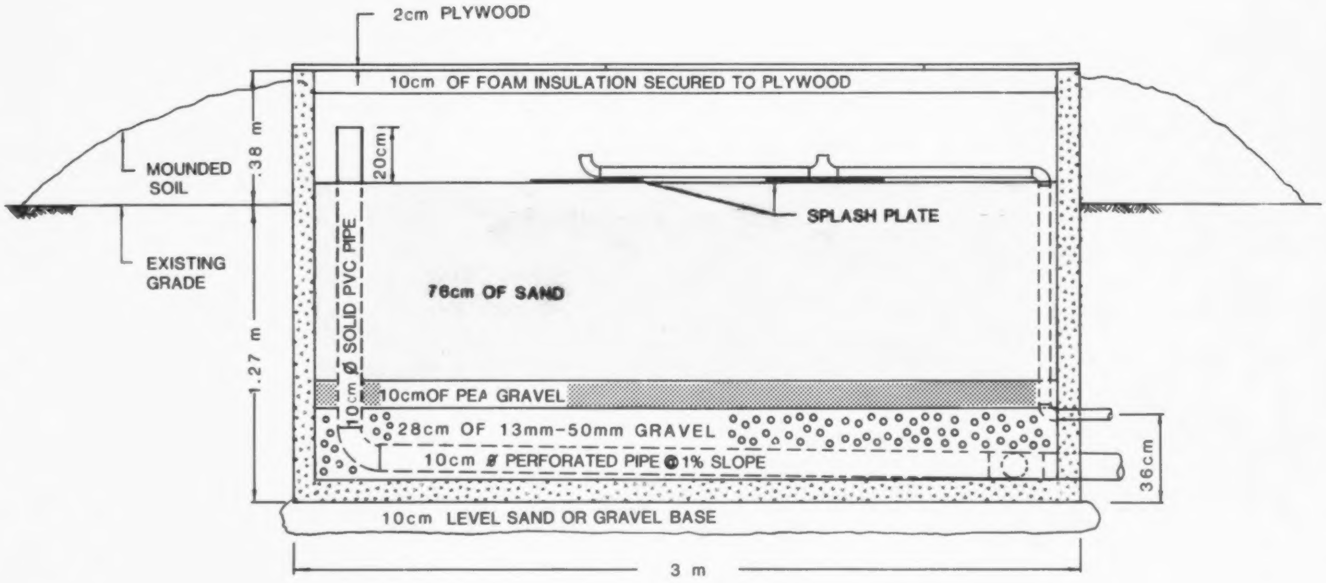

2cm PLYWOOD
10cm OF FOAM INSULATION SECURED TO PLYWOOD
MOUNDED SOIL
EXISTING GRADE
.38 m
1.27 m
20cm
10cm Ø SOLID PVC PIPE
SPLASH PLATE
76cm OF SAND
10cmOF PEA GRAVEL
28cm OF 13mm-50mm GRAVEL
10cm Ø PERFORATED PIPE @ 1% SLOPE
36cm
10cm LEVEL SAND OR GRAVEL BASE
3 m

Figure 26. Sand filter cross-sectional view.

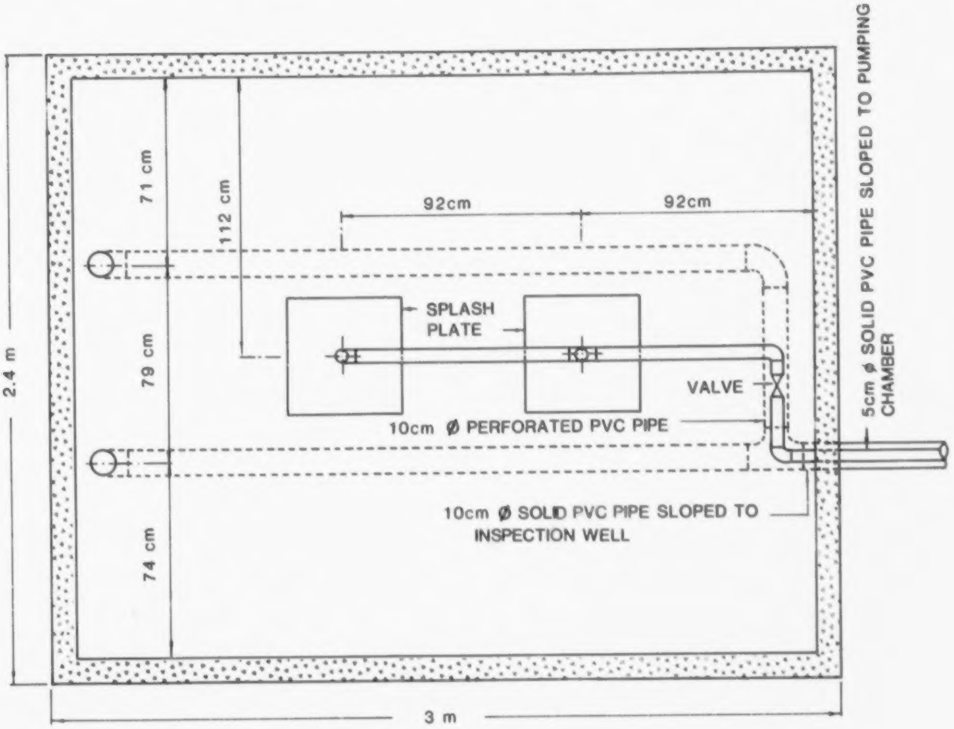

5cm ø SOLID PVC PIPE SLOPED TO PUMPING CHAMBER
71 cm
112 cm
92cm
92cm
SPLASH PLATE
2.4 m
79 cm
VALVE
10cm ø PERFORATED PVC PIPE
10cm ø SOLID PVC PIPE SLOPED TO INSPECTION WELL
74 cm
3 m

Figure 27. Sand filter plan view.

Full-Scale Intermittent Sand Filter--

Intermittent sand filters were installed at two homes in Hamilton County, Ohio, to evaluate the effectiveness and reliability of the filters in an area where sand filters are not commonly used. A research objective of this study was to determine the effect of different levels of wastewater pretreatment on the performance of the filters. Pretreatment was provided by a septic tank at one of the locations and an aerobic unit at the other. Operation and maintenance requirements were also evaluated. This report summarizes these efforts. A detailed report of this study is presented in Appendix F.

The intermittent sand filters used in this study were designed and installed using conventional techniques (USEPA, 1980). Specific design criteria are presented in Table 16. Influent and effluent composite samples from each filter were collected twice a week for 18 months and were analyzed for common wastewater constituents.

The hydraulic loading rates for each filter varied during the study, with an average loading rate for the filter receiving septic tank effluent of 150 lpd/m^2, and 49 lpd/m^2 for the last 11 months of the study for the filter receiving aerobic unit effluent. The first 7 months of data from this filter were not included in this summary because of plumbing difficulties; in addition, new occupants moved into the home during that period.

TABLE 16. INTERMITTENT SAND FILTER DESIGN AND LOADING RATE DATA

Item	Preliminary treatment provided by Septic tank	Aerobic unit
Period of operation	Dec 1981-Sep 1983	Oct 1982-Sep 1983
Size of sand filter	2.2 m x 2.8 m	2.2 m x 2.8 m
Surface area of filter	6.3 m^2	6.3 m^2
Depth of media:		
Sand	76 cm	76 cm
Pea gravel	10 cm	10 cm
Gravel	28 cm	28 cm
Characteristics of sand:		
Effective size	0.44 mm	0.44 mm
Uniformity coefficient	2.5	2.5
Design hydraulic loading rate	162 lpd/m^2	326 lpd/m^2
Actual hydraulic loading rate	150 lpd/m^2	49 lpd/m^2
Dosing quantity (volume/dose)	283 l	254 l
Dosing frequency (doses/day)	3.3	1.2

Septic tank/sand filter system--The influent and effluent data for the septic tank/sand filter system are presented in Table 17. The concentrations of all of the measured parameters in the septic tank effluent were similar to concentrations typically reported in the literature (USEPA, 1978).

Adequate removal of biochemical oxygen demand (BOD_5) occurred through the septic tank/sand filter system. All of the effluent samples tested met the Ohio Administrative Code standard of a mean BOD_5 of 20 mg/l for discharging treated wastewater off of private property (Ohio Department of Health, 1977). Sauer (1975) also found low BOD_5 concentrations in the effluent from sand filters typical to those in this study dosed with septic tank effluent. Total suspended solids (TSS) were reduced an average of 68% through the filter, but only 89% of the samples tested met the Ohio Administrative Code of a mean TSS concentration of 40 mg/l (Ohio Department of Health, 1977). Other researchers have found similar reductions (Sauer, 1975).

TABLE 17. EFFLUENT QUALITY DATA, SEPTIC TANK/SAND FILTER SYSTEM

Parameter	Septic tank effluent Apr 1982-Sep 1983	Sand filter effluent Apr 1982-Sep 1983
BOD_5, mg/l		
Mean	127	4
Range	10-285	2-13
No. of samples	109	68
COD, mg/l		
Mean	310	76
Range	21-571	4-463
No. of samples	104	75
Suspended solids, mg/l		
Mean	53	17
Range	20-132	1-130
No. of samples	111	73
Fecal coliforms No. per 100 ml		
Geometric mean	$2.19\text{x}10^5$	$1.62\text{x}10^3$
Range	$4.90\text{x}10^0\text{-}4.9\text{x}10^7$	$1.34\text{x}10^0\text{-}7.08\text{x}10^6$
No. of samples	111	82
Total-N, mg/l		
Mean	41.5	37.5
Range	6.9-67.8	3.2-113
No. of samples	111	82
Total-P, mg/l		
Mean	19.5	14.1
Range	1.2-39.7	1.8-32.6
No. of samples	116	82

The chemical oxygen demand (COD) concentration was reduced an average of 75% through the filter, but the level of treatment was not consistent.

Fecal coliforms were reduced an average of two orders of magnitude through the sand filter receiving septic tank effluent. However, fecal coliform reduction was not sufficient to meet the surface water standards for water contact recreation use of 200 fecal coliforms per 100 ml (Federal Water Pollution Control Administration, 1968), and disinfection of the sand filter effluent would be required to achieve this standard. Sauer (1975) also found poor fecal coliform reduction through a sand filter similar to that in this study.

An average of 28% of the total phosphorus was removed through the sand filter. This performance is consistent with other sand filters studied where total phosphorus removal ranged from 20 to 30% (Sauer, 1975).

Very little total nitrogen in the septic tank effluent was removed through the sand filter, but nitrification was virtually complete. This is similar to what other studies have found (Sauer, 1975). The average $NO_3^- + NO_2^-$-N concentration of 35.3 in the sand filter effluent exceeded the safe drinking water standard of 10 mg-N/l (USEPA, 1974).

Aerobic unit/sand filter system--The influent and effluent data for the aerobic unit/sand filter system are presented in Table 18. The TSS and fecal coliform concentrations in the aerobic unit effluent were similar to concentrations typically reported in the literature for aerobic unit effluents, while the other parameters measured in this study were considerably lower (USEPA, 1978).

As seen in the septic tank/sand filter system, all of the aerobic unit/sand filter effluent samples met the Ohio Administrative Code standard of a mean BOD_5 of 20 mg/l for discharging treated wastewater off of private property (Ohio Department of Health, 1977). The sand filter effluent had a low COD concentration, but this may have partially resulted from the low influent COD concentration.

TABLE 18. EFFLUENT QUALITY DATA, AEROBIC UNIT/SAND FILTER SYSTEM

Parameter	Aerobic unit effluent Oct 1982-Sep 1983	Sand filter effluent Oct 1982-Sep 1983
BOD_5, mg/l		
Mean	28	2
Range	2-94	1-7
No. of samples	44	53
COD, mg/l		
Mean	63	43
Range	4-161	12-96
No. of samples	70	57
Suspended solids, mg/l		
Mean	38	24
Range	3-93	2-92
No. of samples	62	62
Fecal coliforms, No. per 100 ml		
Geometric mean	5.89×10^3	1.23×10^2
Range	$2.29 \times 10^0 - 5.37 \times 10^5$	$1 \times 10^0 - 9.12 \times 10^4$
No. of samples	72	62
Total-N, mg/l		
Mean	17.8	16.7
Range	8.2-35.1	8.5-32.7
No. of samples	75	61
Total-P, mg/l		
Mean	5.3	2.8
Range	<0.1-15.9	<0.1-11.2
No. of samples	78	66

Total suspended solids were reduced through the sand filter receiving aerobic unit effluent, but the decrease was not as great as the decrease that occurred through the filter receiving septic tank effluent, or as the decreases reported by other studies (Sauer, 1975). The overloading problems that occurred during the first 7 months of this study may have affected the TSS treatment (Appendix F).

In this study, it appeared that a greater number of fecal coliforms were removed through the sand filter receiving aerobic unit effluent than were removed through the sand filter receiving septic tank effluent. Sauer (1975) also found this. However, 31% of the samples exceeded the surface water standards of 200 fecal coliforms per 100 ml, and disinfection of this effluent is recommended. Sauer (1975) also recommended disinfection of sand filter effluent.

Approximately 10% of the total nitrogen in the aerobic unit effluent was removed through the sand filter, and most of the total nitrogen in the sand filter effluent was NO_3^--N, which is expected.

A large reduction of total phosphorus occurred through the sand filter, but because the Total-P in the aerobic unit effluent was much lower than other values reported in the literature (USEPA, 1978), this reduction is probably not representative of the performance of a typical intermittent sand filter treating aerobic unit effluent.

In this study, the intermittent sand filter produced a high quality effluent with respect to BOD_5, suspended solids, and COD regardless of the type of pretreatment. The effluent from the filters usually met the State of Ohio discharge standards for BOD_5 and suspended solids. Neither the septic tank nor the aerobic unit were able to achieve this standard without the sand filters.

The sand filters did not require extensive servicing during the study. The only servicing required was the removal of a 13 mm thick crust from a section of the sand filter receiving aerobic unit effluent. A distribution problem resulted in unequal loading of the filter, and after 10 months of operation, a crust formed on the area that received the greatest volume of effluent. The crusted sand was raked and the distribution problem was corrected. A crust did not develop during the remaining 10 months of the study. The level of maintenance required was unexpectedly low; intermittent sand filters having similar media characteristics and organic and hydraulic loading rates have been reported to require some type of servicing every 6 months (USEPA, 1980).

Table 19 gives the total annual costs for the wastewater treatment systems in this study. Because previous research has found that an intermittent sand filter treating effluent from a home aerobic unit can be hydraulically loaded at twice the rate of a similar filter treating septic tank effluent (USEPA, 1980), the cost of the aerobic unit/sand filter system listed in Table 19 is based on a filter one-half the size of the sand filter treating septic tank effluent in this study. Even with a reduction in filter size, the capital and operation and maintenance costs for the aerobic unit/sand filter system were greater than the costs of the septic tank/sand filter system. The higher costs, amounting to an additional $214.00 per year over a 20-year life are directly attributed to the capital costs of the aerobic unit, and to the substantially higher operation and maintenance costs resulting from energy consumption and aerator replacement. Because of the significantly higher costs of the aerobic unit pretreatment system and because pretreatment did not appear to affect effluent quality with respect to BOD_5, suspended solids, and COD, the septic tank/sand filter system is recommended over the aerobic unit/sand filter system.

TABLE 19. ANNUAL MATERIAL, OPERATION, AND MAINTENANCE COSTS

Item	System costs, 1982 Dollars* Aerobic unit and sand filter	 Septic tank and sand filter
Septic tank	---	$ 72†
Aerobic unit	$359§	---
Pumping chamber/pump	158**	158**
Sand filter, 6.3 sq m	---	153††
Sand filter, 3.2 sq m	80††	---
Total annual costs	$597	$383

* Material costs, amortized at 10% over 20 years, plus O&M costs. The cost of labor for the installation of the systems is not included.
† O&M costs include tank pumping once every 3 yr.
§ O&M costs include annual tank pumping, 550 kW-hr of electricity each year at $0.09 per kW, and aerator replacement every 10 yr.
** Assumes replacement of the effluent pump every 10 yr.
†† Assumes 5 hr of labor for maintenance/yr, at $6.00/hr.

Laboratory-Scale Intermittent Sand Filters--

Two laboratory studies were performed to better define the variable sand filter design criteria. One study evaluated the effect of the hydraulic loading rate on the performance of sand columns used to simulate full-scale intermittent sand filters. The second study evaluated the recirculation ratio on the recirculating sand filter, a modified intermittent sand filter. Summaries of these studies are presented in this section. The complete reports on these studies are presented in Appendices G and H, respectively.

Hydraulic Loading Rate Effects on Intermittent Sand Filter Performance--The effect of hydraulic loading rate on the performance of sand columns that simulated full-scale intermittent sand filters was evaluated. A total of 10 simulated sand filter columns were used to test, in duplicate, 5 hydraulic loading rates ranging from 20 to 162 lpd/sq m, the range of loading rates commonly recommended for intermittent sand filters (USEPA, 1980). The sand filter columns had a surface area of

81 sq cm. Septic tank effluent was applied to the columns at the loading rates and application volumes presented in Table 20.

Septic tank effluents and sand filter effluents were collected two or three times a week over a 6-month period and analyzed for common wastewater constituents. Gas samples were collected at five depths in the columns on five different occasions and analyzed by gas chromatography.

TABLE 20. COLUMN LOADING RATES AND APPLICATION VOLUMES

Columns	Hydraulic loading rate, lpd/sq m	Application vol. per cycle, ml	Application vol. per day, ml
1A & 1B	20	55	165
2A & 2B	51	138	413
3A & 3B	81	220	661
4A & 4B	122	330	990
5A & 5B	162	440	1320

Table 21 presents the average influent and effluent concentrations of the analyzed parameters. The TSS concentration of the sand filter influent was lower than concentrations typically reported in septic tank effluents (USEPA, 1978). The sand filter effluent for all of the hydraulic loading rates studied had low TSS concentrations. The TSS reduction seen in this study was much greater than that observed in other column studies with similar sand sizes (USEPA, 1978). This may have resulted partially from the low influent concentrations.

TABLE 21. MEAN SAND FILTER COLUMNS INFLUENT AND EFFLUENT CONCENTRATIONS, MG/L

Parameter	Influent	20 lpd/sq m 1A	 1B	51 lpd/sq m 2A	 2B	81 lpd/sq m 3A	 3B	122 lpd/sq m 4A	 4B	162 lpd/sq m 5A	 5B
TSS	24	2	3	3	3	3	2	3	3	2	1
Fecal coliforms*	4.8	0	0	0	0	0	0	0.3	0.8	2.7	3.0
Total-P	11.4	4.0	3.5	6.6	6.7	7.6	7.7	8.5	8.6	9.0	8.9
COD	226	39	29	28	27	30	28	30	28	30	30
NH_3-N	66.6	0.6	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.2	0.1
TKN	74.2	3.3	1.6	1.4	1.4	1.4	1.4	1.3	1.3	1.5	1.5
NO_3-N	0.2	95.5	96.8	86.0	86.3	83.9	81.9	82.0	81.4	82.0	76.6
Total-N	74.5	98.4	98.4	87.5	86.7	85.0	83.3	83.6	82.8	83.5	81.3

* Log, per 100 ml.

Fecal coliform numbers were reduced through the sand filter columns, and the amount removed varied with the hydraulic loading rate. At the 20-, 51-, and 81-lpd/sq m loading rates, the fecal coliform concentrations in the sand filter column effluents were below the detectable limit of the test, resulting in a 5-log reduction. At the 122-lpd loading rate, fecal coliforms were detected in the effluent, but were still in very low concentrations. At the highest loading rate, 162 lpd/sq m, fecal coliform numbers in the sand filter effluent were reduced by 2 logs, but the concentration exceeded the standard of 200 fecal coliforms per 100 ml for recreational water contact (Federal Water Pollution Control Administration, 1968). With the exception of the highest loading rate, a greater reduction in fecal coliform numbers was seen in this study compared with a similar column study with a loading rate of 200 lpd/m^2 in which a 2.5-log reduction was seen (USEPA, 1978). These data suggest that lower hydraulic loading rates may reduce the amount of fecal coliforms passing through the filter. This would be an important consideration in areas where effluent disinfection is not required and possible contamination by fecal coliforms in the receiving stream is a concern.

There was also a relationship between Total-P removal and hydraulic loading rate in this study. Higher levels of Total-P reduction occurred at the lower loading rates (Figure 28). The Total-P removal efficiencies ranged from 70% in the columns with the lowest loading rates to 20% in the columns with the highest loading rates. The reduction seen in the higher loading rate was similar to removal seen in other column studies (USEPA, 1978).

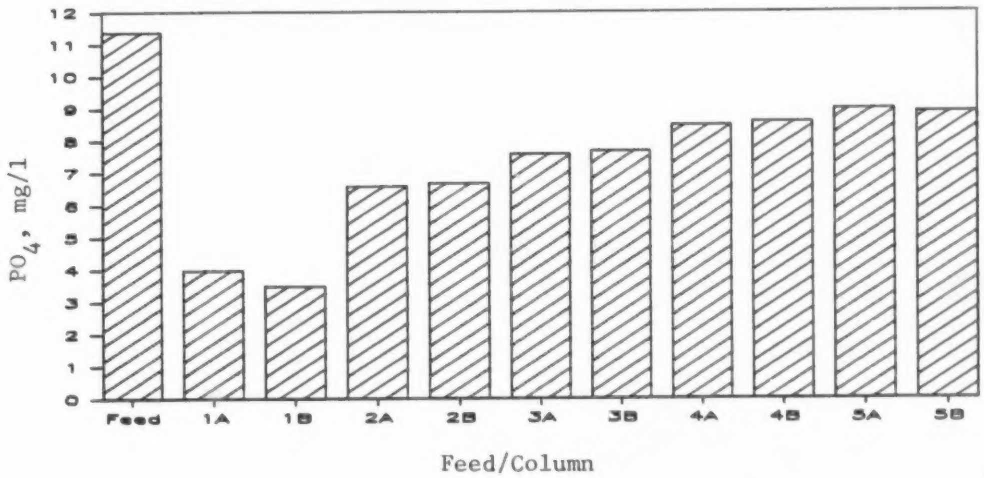


Figure 28. Average influent and effluent total phosphorus concentrations from sand filter columns.

The average sand filter influent COD concentration was within the range typically reported for septic tank effluent (USEPA, 1978), although wide daily fluctuations occurred. These variations had little effect on the COD quality of effluent COD from the sand filter columns, and reductions of COD ranged from 82 to 88%. There was no apparent difference in COD removal among the five hydraulic loading rates.

Influent total nitrogen (Total-N) concentrations were higher than typically reported for septic tank effluent (USEPA, 1978). Nitrification was virtually complete through the columns, which is typical (USEPA, 1978), but for the entire 215 days of this study, the NO_3^--N concentration in the effluent from each of the columns was higher than the influent Total-N concentration. Total nitrogen gains through the columns ranged from 9 to 32%, with the greatest increases occurring in the column with the lowest loading rate. Previous research on intermittent sand filters has shown either a net Total-N decrease or no change at all (USEPA, 1978). Although the reasons for these net nitrogen increases are unknown, it was found that the NO_3^--N analysis had poor accuracy, and could account for as much as a 10 to 15% increase in the Total-N.

The gases, carbon dioxide (CO_2), oxygen (O_2), and nitrogen (N_2), were analyzed to locate anaerobic areas within the column profile and to determine the overall gaseous environment within the sand column. The concentration of N_2 was similar to that of ambient air and was not affected by the hydraulic loading rate or by depth in the sand filter column. The concentrations of both O_2 and CO_2 were affected by both the hydraulic loading rate and by the depth in the column. The higher loading rates had a lower percentage of O_2 and a higher percentage of CO_2 than at the lower loading rates, and O_2 levels decreased while CO_2 levels increased with depth in the sand columns. These concentrations resulted from increased biological activity when more waste was applied and decreased effectiveness of sand aeration at lower depths in the filter. Anaerobic conditions were never observed in these sand filter columns.

In summary, the hydraulic loading rate did not affect effluent quality for TSS or COD for the intermittent sand filter columns in this study, but did affect Total-P and fecal coliforms, which showed significantly less removal at higher loading rates. Total nitrogen showed an inexplicable net gain through all of the columns, with the lowest hydraulic loading rate showing the greatest gain.

These data suggest that sand filters should be designed with lower hydraulic loading rates, perhaps between 50 and 80 lpd/sq m, to reduce the possibility of fecal coliform passing through the filter, and to improve the removal of Total-P.

Recirculation ratio effects on the recirculating sand filter--A modified intermittent sand filter, the recirculating sand filter (RSF), was developed to provide satisfactory treatment of wastewater effluent without the odor potential of an intermittent sand filter (Hines and Favreau, 1975). These filters use a recirculation (dosing) tank between the pretreatment unit and sand filter with provision for return of some of the filtered effluent to the recirculation tank (Figure 29). Thus, the effluent that is applied to the sand filter is diluted with previously filtered effluent.

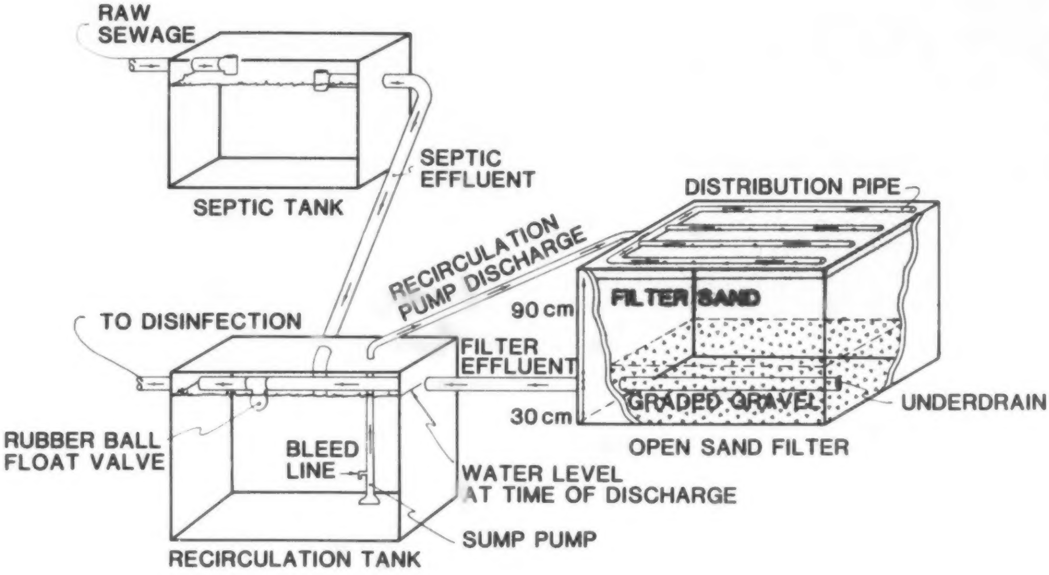

RAW SEWAGE
SEPTIC TANK
SEPTIC EFFLUENT
DISTRIBUTION PIPE
RECIRCULATION PUMP DISCHARGE
FILTER SAND
90 cm
TO DISINFECTION
FILTER EFFLUENT
GRADED GRAVEL
UNDERDRAIN
30 cm
OPEN SAND FILTER
RUBBER BALL FLOAT VALVE
BLEED LINE
WATER LEVEL AT TIME OF DISCHARGE
SUMP PUMP
RECIRCULATION TANK

Figure 29. Schematic of a typical recirculation sand filter (from Hines and Favreau, 1975. Figure 1, p. 132).

Although the RSF has been shown to be effective in treating wastewater and reducing odor (Hines et al., 1977), the most efficient recirculation ratio for the RSF has not been determined. The recirculation ratio of recirculated wastewater to raw wastewater must be in the range of 3 to 1 or greater to maintain aerobic conditions and to minimize odor, but it should be 5 to 1 or less to prevent the filter from being too wet, causing plugging or ponding (Teske, 1980). In this study, effluents with recirculation ratios (recirculated wastewater to raw wastewater) of 3 to 1 (system 1), 4 to 1 (system 2), and 5 to 1 (system 3) were dosed onto three simulated intermittent sand filter columns to determine the optimum recirculation ratio. The 15-cm diameter Plexiglass laboratory columns had a surface area of 180 sq cm.

A loading rate of 122 l/day m^2 based on the raw sewage flow was used in this study. This resulted in a design flow of 92 ml/hr. Septic tank effluent samples and sand filter column effluent samples were collected twice per week for a total of 11 weeks. The samples were analyzed for common wastewater constituents.

Table 22 presents the average concentration and percent removal obtained in the three systems in this study for the parameters tested. The average COD, TKN, and TSS concentrations in the septic tank effluent were higher than values typically reported in the literature for septic tank effluent (USEPA, 1978). Despite the high influent concentration, COD was reduced by all of the systems in this study, with percent removal ranging from 88 to 91%. Other sand filters have been shown to remove high concentrations of COD (USEPA, 1978).

TABLE 22. SEPTIC TANK EFFLUENT AND COLUMN RECIRCULATING SAND FILTER EFFLUENT DATA

Parameter	Septic tank effluent	System 1	Removal (%)	System 2	Removal (%)	System 3	Removal (%)
COD (mg/l)							
Average	686	83	88	66	90	64	91
Range	246-1932	19-171		15-144		18-164	
TKN (mg/l)							
Average	132	40	70	25	81	26	80
Range	55-387	2-100		1-86		1-96	
TSS (mg/l)							
Average	142	18	88	13	91	14	90
Range	70-240	5-115		5-105		5-90	
pH							
Range	6.6-7.7	6.6-7.6		6.8-7.5		6.9-7.6	

Reduction in TKN concentration was seen through all three systems in this study, but there was still a high TKN concentration present in the effluent. Normally, nitrification is almost complete through sand filters provided the filter remains aerobic (USEPA, 1980). The high TKN concentrations in the effluent in this study indicate that complete nitrification did not occur and that perhaps aerobic conditions were not always present in the filters. The effluent TKN concentrations were quite variable, however, and complete nitrification occasionally occurred. Others have found 98% removal of TKN through recirculating sand filters (Ronayne et al., 1982).

Total suspended solids were also effectively removed through the three systems. However, the average effluent TSS concentrations were somewhat higher than other reported recirculating sand filter effluent concentrations of 3 to 12 mg/l (Hines and Favreau, 1975; Ronayne et al., 1982; Teske, 1980). The high septic tank effluent TSS concentration may have contributed to the slightly higher sand filter effluent concentrations.

No difference was found in the pH values of the septic tank effluent and the three recirculating sand filter effluents.

Statistical comparison among the three systems in this study showed that treatment of septic tank effluent COD, TKN, and TSS was not affected by the recirculation ratio.

Operation and maintenance requirements for recirculating sand filters are similar to the requirements for other sand filters, and include normal pump maintenance, periodic raking and leveling of the sand, and removal and replacement of the top several centimeters of sand if a buildup of solid material occurs (Teske, 1980).

Upflow Gravel Filters and Subsurface Gravel Beds

Upflow gravel filters and subsurface discharging gravel beds were also evaluated as effluent polishing units. The upflow gravel filters in this study were manufactured by Jet, Inc., specifically to treat aerobic unit effluent. Aerobic unit effluent flowed by gravity into the precast concrete tank that housed the filter media and was channeled to the bottom of the tank through an influent riser. The effluent flowed upward from the bottom of the filter, through the 15 cm of 6.4- to 9.5-mm smooth silica stone media, and into the effluent discharge trough (Figure 30). According to the manufacturers, the primary method of treatment in the filter is biological (Jet, Inc., 1978). A biological slime grows on the filter media, and as the wastewater flows through, organic material is trapped by the slime. The slime uses the trapped material as food, thus purifying the wastewater.

Subsurface gravel beds have been installed in southwestern Ohio to further treat effluent from aerobic unit-upflow gravel filter systems. These treatment beds are required following aerobic unit-upflow gravel filter systems in Clermont County, Ohio, and were designed by members of the Department of Health in that county. According to the Department of Health, one of the potential benefits of the discharging subsurface gravel beds is that the beds act as detention basins where additional biological treatment and the settling of suspended solids can occur. The beds are typically installed by excavating a 28-sq-m hole to a level bottom depth of 76 cm. Thirty centimeters of 1.9- to 3.8-cm gravel is placed in the hole and a distribution loop of 10.2-cm diameter perforated pipe is laid above the gravel. Additional gravel is then placed around the distribution loop. A straw layer separates the gravel bed from the soil backfill. Effluent is discharged through an overflow connected to the perforated pipe (Figure 31).

Two full-scale, operating aerobic unit-upflow gravel filter-subsurface gravel bed systems were evaluated to determine whether the effluent produced by these wastewater polishing units was of sufficiently high quality to meet or exceed surface discharge requirements for the State of Ohio. The two systems had been operating for 3 to 3.5 yr prior to this study. The upflow gravel filters each had a surface area of approximately 1.24 sq m with 6- to 9-mm smooth silica stone as filter media. The subsurface gravel beds were of standard design. The average daily water use at Residence 1 was 709 lpd and 1474 lpd at Residence 2.

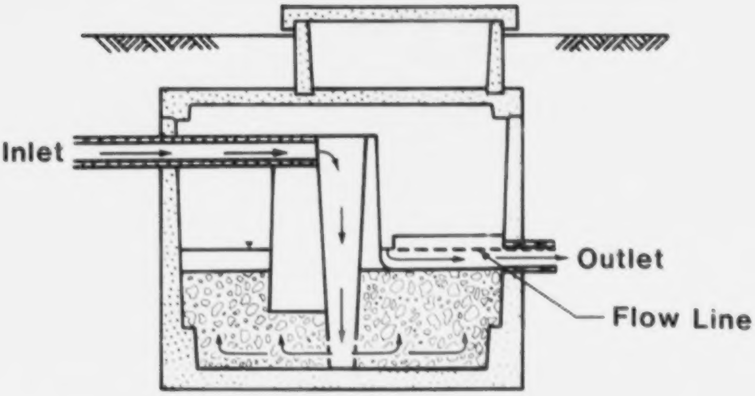

Inlet
Outlet
Flow Line

Figure 30. Diagram of an upflow gravel filter.

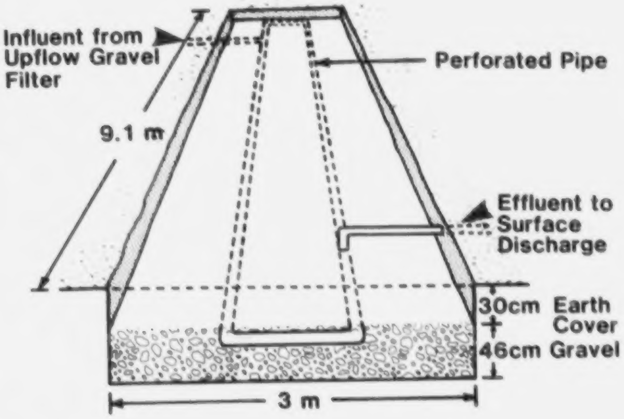

Influent from Upflow Gravel Filter
Perforated Pipe
9.1 m
Effluent to Surface Discharge
30cm Earth Cover
46cm Gravel
3 m

Figure 31. Diagram of a typical subsurface discharging gravel bed.

Influent and effluent grab samples were collected from the systems twice a week for 16 weeks. Samples were collected from the point where the aerobic unit effluent entered the upflow gravel filter, from the discharge trough of the upflow gravel filter, and from the subsurface gravel bed discharge pipe. The samples were analyzed for common wastewater constituents. This report is a summary of the findings. Further information is presented in Appendix I.

Table 23 presents the aerobic unit, upflow gravel filter, and subsurface gravel bed effluent concentrations for the measured parameters. The average BOD_5 concentration was reduced by all of the polishing units. However, the effluent from both the upflow gravel filters and the subsurface gravel beds had a wide range of BOD_5 concentrations, and while all of the samples from Residence 1 met the Ohio Administrative Code standard of a BOD_5 of less than 20 mg/l (Ohio Department of Health, 1977), only 81% of the samples from Residence 2 met the standard. Others have found average BOD_5 concentrations ranging from 11 to 45 mg/l from effluent from upflow gravel filters following aerobic units (Glasser, 1975; Jet, Inc., 1978).

TABLE 23. POLISHING UNIT EFFLUENT CONCENTRATIONS*

	Residence 1			Residence 2		
Parameter	Aerobic unit	Upflow filter	Subsurface bed	Aerobic unit	Upflow filter	Subsurface bed
BOD_5	27	16	7	31	27	11
TSS	39	17	8	62	55	20
COD	95	67	38	121	106	63
Fecal coliforms	32,000	7900	1300	7900	7900	3200
Total-N	17.3	16.4	10.3	33.3	31.4	22.7
NO_2+NO_3-N	9.3	6.9	2.2	3.7	2.5	2.1
Total-P	10.6	10.4	9.9	10.4	10.0	8.2

* All values are mean concentrations expressed as mg/l, with the exception of fecal coliforms, which are geometric mean values expressed as numbers of organisms per 100 ml.

The mean TSS concentration in the effluents from all the polishing units except the upflow gravel filter at Residence 2 met the Ohio Administrative Code TSS standard of 40 mg/l (Ohio Department of Health, 1977). The range of TSS effluent concentrations was large in all the units studied, however, and only the effluent from the subsurface bed at Residence 1 had a concentration of less than 40 mg/l in 100% of the samples tested. Average SS concentrations of 10 and 83 mg/l from upflow gravel filters following aerobic units have been reported (Glasser, 1975; Jet, Inc., 1978).

The two upflow gravel filters and subsurface gravel beds produced effluent with variable, reduced COD concentrations, and the effluent from the subsurface gravel bed had a lower average COD concentration than the upflow gravel filters.

Fecal coliforms were reduced in numbers through all the wastewater polishing units, but the mean number still exceeded the surface water standard for recreational use of 200/100 ml (Federal Water Pollution Control Administration, 1968). Therefore, it is recommended that all surface-discharged effluent be disinfected.

Total nitrogen concentrations were reduced through all the polishing units. Little nitrification occurred through either the upflow gravel filter or the subsurface gravel beds. Because effluent in both of these units remained aerobic, the reason for the low rate of nitrification is not known.

Little reduction of Total-P occurred through the upflow gravel filters or discharging gravel beds.

The upflow gravel filters and subsurface gravel beds in this study received no maintenance prior to this study, nor did they receive any maintenance during the time period of sampling, even though the manufacturer advises routine backwashing of the upflow filters at 6-month intervals. Interviews with the homeowners of these systems revealed that there had been no operational problems during the 3.5 yr that the units had been in service.

Construction costs and operation and maintenance (O&M) costs for the upflow gravel filters and subsurface gravel beds are presented in Table 24. The subsurface gravel bed added an additional $40.00/yr to the annual cost of the systems.

TABLE 24. CONSTRUCTION, OPERATION AND MAINTENANCE COSTS--1981 DOLLARS

Treatment system	Construction costs	Annual O&M*	Total annual costs†
Aerobic unit - Upflow filter	2900	140	285
Aerobic unit - Upflow filter - Subsurface gravel bed	3700	140	325

* Assumes pumping of the aerobic unit every 3 yr, an additional 5 hr of maintenance per year at $10/hr, 550 kW hr of electricity/yr, and aerator replacement every 10 yr.

† Construction costs divided by a 20-yr useful life, plus annual O&M costs.

In summary, the upflow gravel filter at Residence 1, which received the lower hydraulic loading rate, produced a higher quality effluent than the filter at Residence 2 with respect to BOD_5, TSS, and COD. The upflow gravel filters reduced the BOD_5 and TSS concentrations from the aerobic unit effluent, but the effluent from these filters did not consistently meet the Ohio Administrative Code standards for the discharging of treated wastewater off of private property.

The subsurface gravel beds in this study did produce a higher quality effluent than the upflow gravel filters, but because of the method of sampling and the short duration of this study, further work is required to determine whether or not there is enough improvement to warrant the use of the beds along with upflow gravel filters.

Discussion of Surface Discharge Treatment Units

The selection of an onsite wastewater treatment system that discharges effluent into a watercourse should be based primarily on system performance and reliability, and as a minimum requirement, any treatment system should meet local effluent discharge standards before cost becomes an influencing factor. Although the upflow gravel filters and subsurface gravel beds were slightly less expensive than the sand filters, they did not produce an effluent that consistently met the local discharge standards. Therefore, a sand filter system is recommended because both the aerobic unit-sand filter system and the septic tank-sand filter system in this study produced high quality effluents for the two parameters for which there are discharge standards. Since the septic tank-sand filter system achieved this level of treatment at a lower annual cost, the cost of installing and maintaining an aerobic unit as a pretreatment unit for the intermittent sand filter was not justified.

The septic tank-intermittent sand filter system, along with disinfection, appears to be a feasible alternative to systems that rely on the soil for disposal in southwestern Ohio.

SYSTEMS THAT DISCHARGE TO THE ATMOSPHERE

Evapotranspiration (ET) and Evapotranspiration/Absorption (ETA) systems dispose of wastewater by evaporation and transpiration to the atmosphere. In ETA systems, some of the wastewater is also disposed of by seepage into the soil. The year-round use of nondischarging ET systems appears to be feasible only in the arid and semiarid portions of the western and southwestern U.S. where evaporation exceeds precipitation during every month of operation (USEPA, 1980).

The design criteria for an ET system are highly site specific, and are based on the estimated flow from the home and the difference between the precipitation rate and the evaporation rate during the critical months of the year (Bennett & Linstedt, 1978; USEPA, 1980).

An ET system typically consists of a sand bed and wastewater distribution piping. An impermeable liner is installed in the ET system. The system functions by raising the wastewater to the upper portion of the bed by capillary action in the sand, and then evaporating the wastewater to the atmosphere. In addition, vegetation transports water from the root zone to the leaves, where it is transpired. In ETA systems, the impermeable liner is omitted (USEPA, 1980).

In 1978, the Hamilton County, Ohio, Department of Health approved the use of ET and ETA systems on an experimental basis at locations where more traditional onsite treatment was not acceptable. From 1978 to 1981, 11 systems were known to be installed in the county. The performance of these systems was evaluated by the Cincinnati Center for Small Community Wastewater Systems Studies during the spring of 1982 and again in the winter of 1985. A summary of the findings is presented here. A complete report is presented in Appendix J.

Information about each system was obtained through interviews with homeowners, health department personnel, plumbers, public water utilities, and onsite inspections. Figure 32 is a cross section of a typical system evaluated in this study. The design was not a standard design (USEPA, 1980), and although the Health Department in Hamilton County, Ohio, issued recommendations for construction, variations were common in the field installations. Except for one system that used a septic tank, treatment by a home aerobic unit preceded each of the ET/ETA systems. A dosing chamber was present in all of the systems and wastewater was pumped from this chamber to the systems.

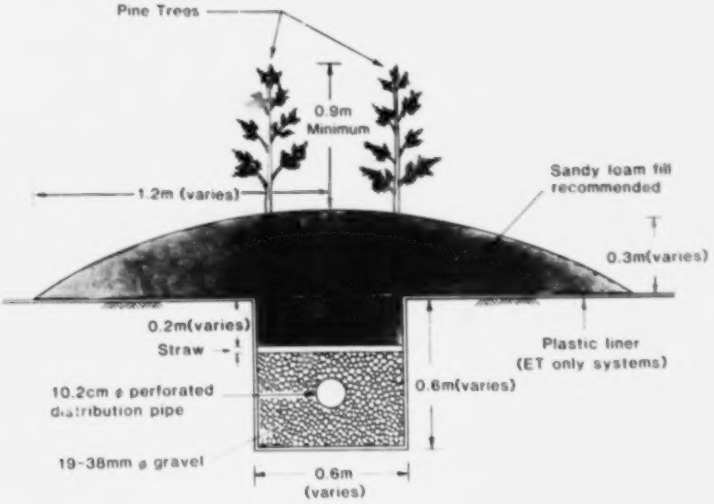

Pine Trees
0.9m Minimum
1.2m (varies)
Sandy loam fill recommended
0.3m(varies)
0.2m(varies)
Straw
Plastic liner (ET only systems)
10.2cm ø perforated distribution pipe
0.6m(varies)
19-38mm ø gravel
0.6m (varies)

Figure 32. Cross section of a typical ET/ETA system installed in Hamilton County, Ohio.

Of the 11 systems evaluated, only 5 were considered to be functioning properly. Characteristics of these systems are presented in Table 25. Sites with surface breakout on or around the mound, and sites modified with overflow pipes were considered to be failing, and characteristics of these systems are given in Table 26. The systems that were working had always worked, and those that were failing had never worked, and there was no apparent change in the performance of the systems during the two time periods of evaluation.

Of the 11 systems evaluated, there was no correlation between water usage and the size of the system, which includes the number of trenches and the dimensions of the soil-filled mound. The systems were installed in 6 different soil series, and although sandy loam fill was specified by the county, clay, loam, silt loam, and clay loam fills were also used. Three of the systems were ETA systems and did not have plastic liners, so soil absorption also occurred. The owners of a fourth system claimed that a liner was installed, but the contractor denied this. The number of living trees on the systems at the time of the evaluation ranged from 9 to 30 and consisted of Scotch and Austrian pine.

Because variation existed in the design and installation of the systems, it is difficult to determine why a system did or did not work. Both groups (failing and properly functioning) included ET and ETA systems, had low and high flow rates, multiple and single trench designs, various mound fill materials, and were installed in suitable and unsuitable soils. Also, the liners in the ET systems may have been torn during construction, allowing soil absorption that could not be quantified, and all modifications may not have been reported, making comparisons among these systems even more difficult.

A system that relies solely on evapotranspiration theoretically cannot work in a humid area like Ohio without considerable design modification and careful management (Bonderant, 1984). At the Agricultural Research Station at Coshocton, Ohio, precipitation exceeds potential evapotranspiration during 6 months of the year, and under such conditions an ET system cannot function because effluent added to the soil will simply add to the surplus moisture (Bonderant, 1984). Therefore, it is likely that the systems in this evaluation that were working either had soil absorption, modifications that were not reported or were not detected by visual observation, and/or very low flows. The only parameter that may have been different between the two groups was the number of trenches in the system. In the systems that were functioning properly, 3 of the 5 systems had multiple trenches, while the remaining 2 only had 1 trench. However, both of these latter 2 systems served small businesses, and 1 of these had low water usage. In the failing systems, only 2 of the 6 systems had multiple trenches, and in 1 of these 2, the system may not have been installed properly because breakout occurred at the middle of the system when the ends of the trenches were still dry. However, there was not sufficient information available to conclude that the number of trenches actually affected system performance.

TABLE 25. CHARACTERISTICS OF PROPERLY FUNCTIONING ET/ETA SYSTEMS IN HAMILTON COUNTY, OHIO

	Systems				
Characteristic	7	8	9	10	11*
Age, yr	5	6.5	7	6.5	7
Water usage, lpd	1360, est.†	280, est.§	600	800	670
Dimensions of mound over trenches l x w x h, m	27x4.9x1.2	20x-x-	30x2.6x0.3	20x4.6x0.5	10x3.7x0
Number of distribution lines	2	1	1	3	3
Liner installed	yes	yes	yes	yes/no**	no
Mound fill material	loam	clay	clay loam	loam	clay loam
Soil mapping unit at site††	Bonnell silt loam	Ava-Urban land complex	Avonburg silt loam	Rossmoyne-Urban land complex	Cincinnati silt loam
Owner modification to system	no	no	no§§	no	no

* This was the only system that disposed of septic tank effluent.
† Water usage estimated from 24 employees at factory, 40 hr/wk.
§ Water usage estimated from 3 employees at service station, open 20 hr/day.
** Homeowner recalled the installation of a liner, but the contractor did not.
†† From Soil Survey of Hamilton County, Ohio (Lerch et al., 1982).
§§ Homeowner said that no modifications had been made, but neighbors thought there had been.

TABLE 26. CHARACTERISTICS OF FAILING ET/ETA SYSTEMS IN HAMILTON COUNTY, OHIO.

	Systems					
Characteristic	1	2	3	4	5	6
Age, yr	3	4.5	6.5	6.5	6.5	6.5
Water usage, lpd	830	1120	510	610	740	750
Dimensions of mound over trenches l x w x h, m	20x3.4x0.5	33x1.5x0.2	30x3.7x0.9	33x4.3x0.9	34x4.3x0.6	30x5.2x0.4
Number of distribution lines	3	1	1	1	1	2
Liner installed	no	yes	yes	yes	yes	no
Mound fill material	sandy loam	sandy loam	clay	clay	clay	silt loam
Soil mapping unit at site*	Eden-Urban and complex	Ava-Urban land complex	Avonburg silt loam	Avonburg silt loam	Avonburg silt loam	Rossmoyne-Urban land complex
Owner modification to system	no	yes	yes	yes	yes	yes

* From Soil Survey of Hamilton County, Ohio (Lerch et al., 1982).

This survey of previously installed ET/ETA systems identified 5 of 11 systems that were apparently handling the applied wastewater. Because variable design, construction, and site conditions existed, factors that contributed to the success of these systems could not be determined. However, because precipitation exceeds potential evaporation for 6 months of the year, systems that rely solely on evapotranspiration for the disposal of effluent should not be considered to be suitable alternatives for year-round use in southwestern Ohio. It is likely that the 5 systems that appeared to be working in this evaluation were not true ET systems and that other factors were involved with their satisfactory performance.

SECTION 7

SITE EVALUATION TECHNIQUES FOR ONSITE WASTEWATER TREATMENT AND DISPOSAL SYSTEMS ON ILLINOIAN TILL SOILS

Evaluation of site and soil suitability is perhaps the single most important aspect of onsite sewage disposal (Hargett et al., 1983). The success or failure of a system depends on an accurate estimation of a soil's ability to absorb, transmit, and treat the wastewater applied. Knowing the soil and site characteristics permits planners to select the most appropriate treatment and disposal system. Detailed procedures for performing site evaluations can be found in USEPA (1980).

Two site and soil characteristics that are very important for a properly functioning soil absorption system are the depth of soil over zones of saturation and the rate of water movement through the soil. Numerous studies have shown that 60 to 120 cm of unsaturated soil is required to remove bacteria and viruses, and nearly all phosphorus, from wastewater to acceptable levels (USEPA, 1978). Also, water must be absorbed and must move through the soil quickly enough to dispose of all the water produced, while moving slowly enough to treat the effluent.

The site evaluation techniques that are typically used to evaluate these two characteristics have recently been criticized. These techniques include the use of soil mottles to predict the depth to seasonal saturation, and the use of soil percolation tests to provide an index of soil permeability.

Soil mottles (spots of contrasting grayish and reddish colors) within a soil profile are commonly used to determine the depth to saturation during those times of the year when a water table is not present. Mottles are formed during alternating periods of oxidation and reduction normally associated with unsaturated and saturated conditions, respectively (Bouma, 1977). Iron and manganese compounds in the soil become soluble when they are reduced and can move with water through the soil profile. The removal of iron by flowing water can permanently change the color of the soil from which the iron was removed to gray, giving it a chroma of 2 or less according to the Munsell color notation (Soil Survey Staff, 1975; Bouma, 1977). After the reduced minerals oxidize, they precipitate and form reddish and black spots. Soil mottles that have chromas of 2 or less are assumed to indicate that the soil horizon that has such mottles is saturated with water at some period of the year when the temperature of the horizon is $5^{\circ}C$ or more (Soil Survey Staff, 1975). More discussion of on this subject can be found in Veneman et al. (1976).

Although soil mottles provide an easy and inexpensive method of judging the soil moisture regime, it has been shown that while mottling may indicate saturation, lack of mottling does not necessarily imply lack of saturation (Bouma, 1977). Seasonally high water tables have been measured in horizons that do not contain mottles with chromas of 2 or less (Daniels et al., 1971; Evans & Franzmeier, 1983; Simonson & Boersma, 1972; Vepraskas & Wilding, 1983), and in some soils, mottles with chromas of 2 or less have been reported in horizons above the actual high water table (Vepraskas et al., 1974).

The percolation test, proposed by Ryon in 1926 (Federick, 1948), is currently used in many areas to provide an index of soil permeability, and is required to some degree in most states to size a soil absorption field. However, the use of the percolation test for this purpose has received much criticism because of its high variability (Barbarick et al., 1976; Derr et al., 1969), the unclear relationship between the percolation rate and seepage bed performance (Healy & Laak, 1973), and the physically ill-defined nature of the test (Bouma, 1971).

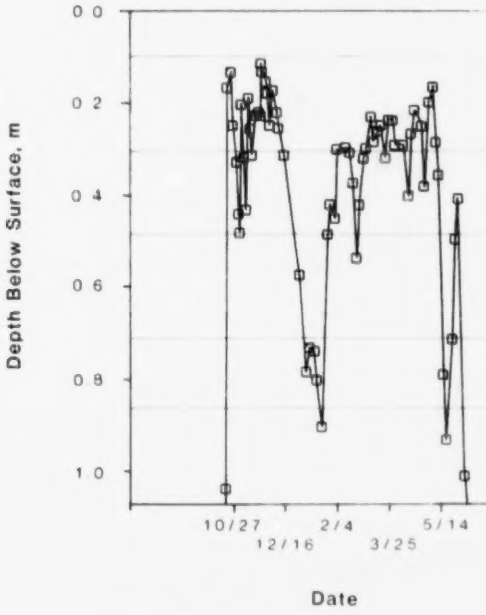
Some studies have proposed using hydraulic conductivity (K) tests to determine the potential of soils for onsite wastewater treatment and disposal rather than percolation tests (Bouma, 1971, 1975). Bouma (1971) concluded that curves of K values for saturated and unsaturated soil were more meaningful than a single percolation rate because the curves have the full range of possible infiltration rates (at unit hydraulic gradient) resulting from different degrees of clogging mat formation.

In this study, the use of soil mottles and percolation tests in site investigations in Illinoian till soils was evaluated at the Cincinnati Center's field research site (Figure 1). The specific objectives were to determine whether present morphological classification schemes using mottles with chromas of 2 or less adequately describe the frequency and duration of the high water table, and to determine whether hydraulic conductivity values and percolation rates accurately describe the hydraulic capacity of Avonburg and Rossmoyne soils to absorb and transmit wastewater. This section presents a summary of these studies. Complete reports of these studies are presented in Appendices A and K.

SOIL MORPHOLOGICAL INDICATIONS OF WETNESS

To assess the use of soil mottles, soil core samples from each groundwater monitoring well location (Figure 2) were described in terms of soil mottles using traditional terminology (Soil Conservation Service, 1981) and Brewer's terminology (Brewer, 1964). The soil colors were then compared with the groundwater table data obtained.

The soil mottling patterns and corresponding groundwater table elevations are presented for a typical monitoring location in Figure 33. In the example, the Ap2 horizon (0.1 to 0.3 m) contained manganese (Mn) nodules and faint silt cutans (coatings) with chromas of 3. The groundwater table was observed within this horizon 33% of the time during 2 yr of data collection. In the BE horizon (0.3 to 0.48 m), Mn nodules and cutans were present along with more distinctive silt cutans with chromas of 3. The groundwater table was observed within this horizon 47% of the time. Although the groundwater table was observed within these two horizons for extended periods of time, the horizons did not contain mottles with chromas of 2 or less, those frequently used to indicate that the soil horizon is saturated with water at some period of the year (Soil Survey Staff, 1975). The presence of mottles with chromas of 3 and Mn nodules and cutans in horizons that do not contain mottles with chromas of 2 or less but are saturated for short periods of time has been observed in other studies (Daniels et al., 1971; Franzmeier et al., 1983; Vepraskas & Wilding, 1983).



Horizon	% of Time Groundwater Observed Within Horizon	Soil Morphological Features
Ap1	0	
Ap2	33	Mn nodules 3-chroma cutans
BE	47	Mn nodules & cutans 3-chroma cutans
2Btgb	51	Mn nodules & cutans Fe cutans 3-chroma cutans 2-chroma mottles & cutans
2Btxgb1	55	Mn nodules & cutans 1-chroma mottles & cutans
2Btxgb2	60	Mn nodules & cutans Fe cutans 2-chroma mottles & cutans

Figure 33. Depth of the observed groundwater table and corresponding soil morphological features for a typical monitoring location in an Avonburg silt loam soil.

Mottles with chromas of 2 were first present in the 2Btgb horizon, at a depth of 0.48 m. The groundwater table was observed within this horizon 51% of the time. These mottles were in the form of clay cutans and as mottles within peds. Ferrans (iron cutans) were also present in this horizon.

The lack of mottles with chromas of 2 or less in horizons that are saturated with water may be the result of periods of saturation that are not associated with periods of reduction. Although the soil pores are filled with water, enough dissolved oxygen may still be present in the water to maintain aerobic conditions. This would keep the process of reduction from proceeding to the extent of reducing Fe(III), a condition necessary for mottle formation (Vepraskas & Wilding, 1983).

In this study, while the Ap2 and BE horizons were saturated for an extended period, the continuous duration of saturation was short because the groundwater table constantly fluctuated through them. Oxygen could constantly be introduced into the horizons, preventing the formation of reducing conditions. However, in the 2Btgb horizon, few fluctuations occurred and the groundwater table was within this horizon for extended periods of time, allowing the formation of reducing conditions.

It is apparent from this and other studies that mottles with chromas of 2 or less are not always formed during saturation, and that in some soils more information should be obtained to accurately describe the soil moisture regime. Several proposals have been developed to include additional morphological features to describe soil moisture. Vepraskas and Wilding (1983) proposed to include those conditions with dominant chromas of 3 on ped exteriors when accompanied by a mottled ped interior with matrix chromas of 4 or more in the aquic moisture regime. Franzmeier et al. (1983) suggested three classes of soil wetness and aeration as related to soil color pattern, including 2-chroma colors to indicate periodic saturation, and 3-chroma colors to also indicate periods of saturation, but less reduced than horizons with 2-chroma colors. Veneman et al. (1976) formed three broad categories of soil moisture regimes, which included horizons with mottling dominated by high chromas in peds, ped mangans, and few Fe mottles that were associated with very short periods of saturation.

The results of this study support the suggestions that additional categories are required to accurately describe the moisture regime for soils such as those in this study.

In summary, soil morphological features such as mottles, cutans, and nodules could generally be used to predict the presence and duration of the high water table in the Avonburg and Rossmoyne soils in this study using extended criteria. However, using only the presence of mottles with chromas of 2 or less would underestimate the height of the actual high water table, possibly making sites that are unsuitable for soil absorption appear to be suitable. Therefore, it is proposed to include other soil morphological features such as mottles with chromas of 3 and Mn and Fe nodules and cutans in site evaluations on Avonburg and Rossmoyne soils to most accurately predict the presence and duration of the high water table in these soils.

HYDRAULIC CONDUCTIVITY

In situ hydraulic conductivity tests and percolation tests were performed at four locations at the field research site (Figure 1). Two of the locations were in the gently sloping Rossmoyne soils and 2 were in nearly level Avonburg soils. The tests performed included the percolation test, an in situ falling-head permeability test, the auger-hole method, and the crust test. These methods were chosen because they are either generally accepted methods for determining hydraulic conductivity, are innovative, or are simple to perform. All of the tests give saturated hydraulic conductivity values, and the crust test determines unsaturated hydraulic conductivity values. All of the tests were performed at a 50 cm depth to correspond with the depth of the infiltrative surface of a conventional soil absorption system.

The hydraulic conductivity values and percolation rates were compared with actual absorption rates that were obtained from 2 conventional trenches installed in an Avonburg silt loam soil. The mechanism used to apply effluent to each of the trenches allowed for an accurate measurement of the volume of liquid absorbed by each trench. See Section 5 for further information on these trenches.

The saturated hydraulic conductivity values obtained from the percolation test, the falling-head permeability test, and the auger-hole method are presented in Table 27. The data were found to be log-normally distributed, and logarithmic transformations were performed on the data so that normal statistics could be applied. The percolation test results were converted to hydraulic conductivity values by the method of Machmeier (1975).

The geometric mean of saturated hydraulic conductivities, found to be a good estimator of average conductivity (Bouwer, 1969), ranged from 4 to 158 cm/day over the four test sites. A wide range of values existed in all of the tests and soils evaluated. Other studies have reported large variability for saturated hydraulic conductivity tests and percolation tests (Barbarick et al., 1976; Bouma, 1971; Derr et al., 1969; Hillel, 1980b).

Analyses of variance and Scheffe's test (SAS Institute, Inc., 1985) were used to test for significant differences among the tests and the 4 sites. No significant difference was found among the tests performed. In terms of saturated hydraulic conductivity values, percolation rates converted to saturated K values by the method of Machmeier (1975) provided values similar to the auger-hole method and the falling-head permeability test. Because of the lack of significant differences among the three tests and the soils, the hydraulic conductivity values were considered to be from one population, and an average hydraulic conductivity value of 33 cm/day was obtained.

TABLE 27. SATURATED HYDRAULIC CONDUCTIVITY VALUES FOR AN AVONBURG AND ROSSMOYNE SOIL

		Hydraulic conductivity		
Soil	Percolation rate (min/cm)	Percolation test	Falling-head test	Auger-hole test
		(cm/day)		
Avonburg 1				
G*	14	28	101	12
Range	8.7-21	19-48	19-657	9.5-19
Avonburg 2				
G	9.8	52	4	32
Range	8.7-12	42-59	1.4-18	12-82
Rossmoyne 1				
G	5.8	96	19	158
Range	3.5-9.4	55-173	6.2-95	68-587
Rossmoyne 2				
G	†	†	4	†
Range			1.4-6.7	

* Geometric mean of the untransformed data. This is equivalent to the antilogarithm of the log-transformed data.

† The test was not successfully performed.

The unsaturated hydraulic conductivity curve generated by the crust test is presented in Figure 34. The curve developed in this study is similar to curves developed in other soils with high clay contents (Bouma et al., 1974). The curve indicates that once the macropores in these soils have drained, which occurs at lower soil moisture tensions, very little flow occurs. Flow becomes nearly neglible at higher soil moisture tensions. Although the crust test provides much information concerning the soil moisture regime, it is not practical to perform on a routine basis because of the length of time and amount of equipment required to perform the test. Therefore, it would not be suitable for routine onsite investigations.

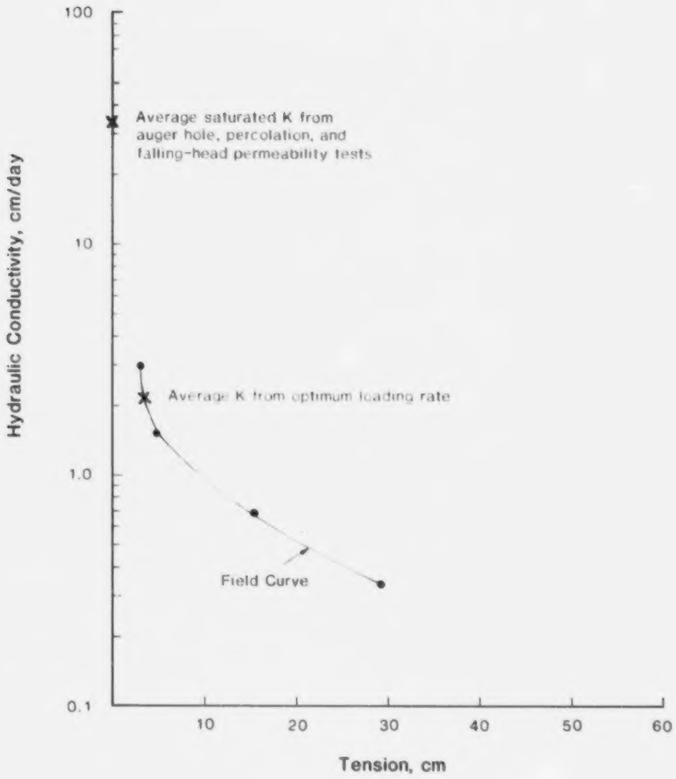


Figure 34. Hydraulic conductivity curve derived in situ with the crust test.

The actual average loading rates of 2 conventional trenches installed in an Avonburg silt loam are presented in Table 28. The loading rates varied considerably throughout the year, being greatest during the late summer months, when the water table and soil moisture content were low and evapotranspiration rates (ET) were high, and lowest during the winter months when the water table and soil moisture content were high and ET was low. A further discussion of the loading rates is presented in Appendix D of this report.

TABLE 28. AVERAGE DAILY ABSORPTION RATES OF CONVENTIONAL TRENCHES INSTALLED IN AN AVONBURG SILT LOAM

	Absorption rate (cm/day)*			
Year	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec
1983	--	--	5.9†	1.1
1984	0	1.9	2.7	0.1
1985	1.1	1.2	1.9§	--

* Based on bottom area of trench.
† Start-up period--includes only September.
§ Includes only July and August.

The mean value of loading rates from April to September 1984, and July to August 1985, was 2.2 cm/day and was taken to be the long-term acceptance rate under optimum climatic conditions for these trenches. This corresponds to a soil moisture tension of 3.5 cm on the K-curve. This low tension indicates that although some resistance to flow was present in the trenches, indicating some degree of clogging mat formation, the soil was still in a near-saturated state.

None of the saturated hydraulic conductivity tests or the percolation test adequately described the actual loading rates. The actual maximum hydraulic loading rate after a 2-yr period was only approximately 6% of the saturated hydraulic conductivities derived from the auger-hole method and the falling-head permeability test, and 1.4% of the percolation rate. Bouma (1971) found that real estimated flow rates were only 0.1 to 8% of the percolation rates.

When a subsurface soil absorption system is designed using the percolation test, the infiltrative surface is sized with the use of tables that correlate application rate to the percolation rate. These application rates have been empirically determined (Bouma, 1971). According to widely recommended tables, the maximum allowable loading rate (based on bottom areas of seepage systems) for the slowest percolation rate in this study is 1.83 cm/day (18.5 lpd/m^2) (USEPA, 1980). This is similar to the long-term acceptance rate during the summer months for the conventional trenches in this study (2.2 cm/day). However, the actual loading rates were much lower during the winter months, and the percolation rate greatly overestimated the absorption capacity of the soil during these time periods. Thus, the percolation test approach for sizing, although a useful tool for some situations, appears to have serious inadequacies for the soils in this study.

USE OF SOIL MORPHOLOGY TO PREDICT THE MOISTURE REGIME OF A SOIL

Recently, many researchers have shown that soil morphology alone can be used effectively to determine site suitability and system size, and that the percolation test, or other hydraulic conductivity tests, can be eliminated altogether (Barbarick et al., 1976; Derr et al., 1969; Hargett et al., 1983). Hargett et al. (1983) provide a thorough discussion on this topic. The U.S. Public Health Service produced data showing that experienced judgment compared favorably with percolation tests (Winneberger, 1974). Several states rely primarily on the soil profile evaluation to determine site suitability for onsite sewage disposal (Long, 1980; Struchtemeyer & Black, 1982; Uebler, 1984).

Although no statistical correlations were made between soil properties and percolation rates in this study, the hydraulic capacity of the soil was inferred from the soil morphological features. The soils had silty clay loam to clay loam subsoil textures, with well-defined prismatic, subangular blocky, and angular blocky structure, high bulk densities, and firm to extremely firm consistencies. Many worm and root channels were present, a fragipan or dense horizon existed, and mottling occurred at a relatively shallow depth, indicating the presence of a fluctuating high water table.

The high clay content of the soil, the firm consistency and high bulk density, the fragipan or dense horizon, and the fluctuating high water table are all properties that hinder water movement. This suggests that soil absorption would be slow in these soils, especially during those times of the year when the water table was high. However, the well-defined structural units and macropores suggest that flow of water could occur relatively rapidly during saturated flow.

The pattern of water movement suggested by the soil morphology was shown to be accurate by the various tests performed in this study. The percolation test and hydraulic conductivity tests showed that saturated flow occurred at a slow, but acceptable rate for wastewater absorption. However, the K-curve generated by the crust test indicated that once the macropores are removed from the flow system, the flow rate becomes very slow. The actual absorption rates showed that the high groundwater table greatly reduced the rate of wastewater absorption.

Because of the large variability and low accuracy associated with hydraulic conductivity and percolation tests, it is possible that detailed soil morphological investigations by skilled soil scientists could be used to predict the hydraulic capacity of a soil as well as the various tests used to predict this. Therefore, it is recommended that K-curves be developed for the dominant soils in an area, and then for each onsite investigation, only a thorough soil morphological description would be needed to accurately size the soil absorption system.

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Appendix A

SHALLOW GROUNDWATER CHARACTERISTICS OF A SMALL ILLINOIAN TILL PLAIN SITE*

Margaret Cashell David Effert

INTRODUCTION

The groundwater table occurs close to the soil surface in many of the soils in southwestern Ohio during all or part of the year. This condition greatly limits these soils for many uses, including wastewater absorption, agricultural practices, and building construction. In order to make wise land use decisions concerning these soils, it is important to determine the frequency and duration of the shallow free water surface, and to characterize the moisture regime of the soil.

The most accurate characterization of the soil-water system is often obtained by the direct measurement of the soil moisture regime. However, because of the length of time and high labor costs associated with this method, the monitoring locations are usually spaced at wide intervals over large tracts of land and groundwater levels are not observed very frequently (Bouma, 1977). Therefore, only generalized information is obtained. More quantitative information about the capacity of a soil to hold and transmit water, about seasonal changes in soil water content and depth to the water table, and about the hydrologic relationships of soils in a landscape is often needed (Harlan and Franzmeier, 1974).

Soil mottles (spots of contrasting grayish and reddish colors) within a soil profile are commonly used to characterize the moisture regime of a soil during those times of the year when a water table is not present. Mottles are formed during alternating periods of oxidation and reduction normally associated with unsaturated and saturated conditions, respectively (Bouma, 1977). Iron and manganese compounds in the soil become soluble when they are reduced and can move with water through the soil profile. The removal of iron by flowing water can permanently change the color of the affected soil to gray, giving it a chroma of 2 or less according to the Munsell color notation (Soil Survey Staff, 1975; Bouma, 1977). After the reduced minerals oxidize, they precipitate and form reddish and black spots. Soil mottles that have chromas of 2 or less are assumed to indicate that the soil horizon that has such mottles is saturated with water at some period of the year when the temperature of the horizon is 5^{o}C or more (Soil Survey Staff, 1975). Many regulatory agencies use these mottles as the legal basis to determine the depth to the high water table during those times of the year when the groundwater table is not present.

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* Abstract in: Agronomy Abstracts. 1985. Amer. Soc. Agron., Inc., Madison, Wisconsin. p. 189.

Although soil mottles provide an easy and inexpensive method of judging the soil moisture regime, it has been shown that while mottling may indicate saturation, lack of mottling does not necessarily imply lack of saturation (Bouma, 1977). Seasonally high water tables have been measured in horizons that do not contain mottles with chromas of 2 or less (Daniels et al., 1971; Evans and Franzmeier, 1983; Simonson and Boersma, 1972; Vepraskas and Wilding, 1983), and some soils have been reported to have mottles with chromas of 2 or less above the actual high water table (Vepraskas et al., 1974).

Other soil morphological features such as iron and manganese nodules and coatings, and mottles with chromas of 3 have also been shown to be associated with a high groundwater table (Franzmeier et al., 1983; Veneman et al., 1976; Vepraskas and Wilding, 1983). These features do not appear to be consistent with all soil types, however.

In this study, direct measurements of the groundwater table, soil moisture content, and soil moisture potential were frequently made at closely spaced intervals over a 2.5-yr period to describe most accurately the soil moisture regime of a 1.2-hectare site used by the Cincinnati Center for Small Community Wastewater Systems Studies for onsite wastewater disposal research. The objectives of this study were to:

1. Establish the baseline groundwater hydrology of the site.
2. Determine climatic and landscape position effects on the soil moisture regime over a small area of land.
3. Determine if present morphological classification schemes adequately describe the frequency and duration of the high water table.

STUDY AREA

The study was conducted on a 1.2-hectare Illinoian till plain site in Clermont County, Ohio (Figure A-1). The north-facing site was the downslope section of a 240-hectare watershed and sloped toward a stream tributary. The southern portion of the site was nearly level, with slopes increasing to a maximum of 12% in the northern section nearest the stream (Figure A-2). The site had been a fallow pasture for at least 10 years prior to the study and the vegetation was kept mowed during the study.

The soils at the site were classified as Avonburg silt loams (Aeric Fragiaqualf, fine-silty, mixed, mesic) and Rossmoyne silt loams (Aquic Fragiudalf, fine-silty, mixed, mesic), two soils of the same catena. These soils formed in 40 to 100 cm of Wisconsinan loess and in the underlying weathered glacial till of Illinoian age. Interbedded Ordovician limestone/shale bedrock occurred within 3 m over the site and commonly was found at less than 2 m.

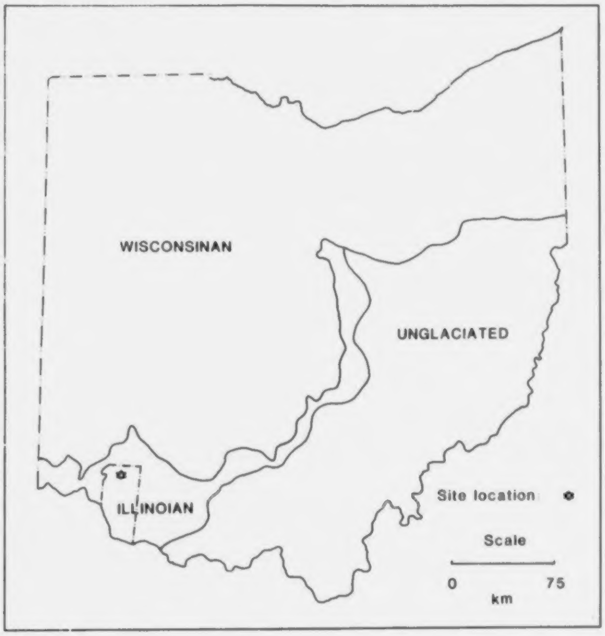


Figure A-1. Location of study area in Clermont County, Ohio.

Typical soil mophological characteristics for these soils are presented in Table A-1, and a typical soil profile is graphically shown in Figure A-3. In general, the subsoils had silty clay loam to clay loam textures and strong, blocky and prismatic structures. A fragipan (a dense soil horizon that can impede downward water movement) was present at a depth ranging from 68 to 100 cm. The fragipan varied in classification properties over the site and in many areas could not be classified as a fragipan. The soils also had high bulk densities in the subsoil horizons, and there was an abundance of macropores and planar voids between the structural units.

The climate of Clermont County is characteristically humid, temperate, and continental (Lerch et al., 1975). Summers are warm and rather humid, and winters are moderately cold (NOAA, 1983). The average annual precipitation received is 102 cm, with precipitation often occurring as snowfall during the winter months. A precipitation maximum occurs during winter and spring with a late summer and fall minimum.

TABLE A-1. SOIL MORPHOLOGICAL CHARACTERISTICS OF AN AVONBURG SILT LOAM SOIL

		Munsell Color, moist							
Horizon	Depth, cm	Matrix	Mottles*	Hand Texture†	Structure§	Bulk Density** g/cc	Consistency††	Boundary§§	Remarks
Ap1	0-10	10YR3/3		sil	mgr	1.24	vfr	cw	
Ap2	10-28	10YR3/3		sil	1m&csbk	1.33	fr	cw	Few to common thin patchy clay films on some ped faces; few fine black nodules.
BEt	28-36	10YR4/4 10YR5/4	7.5YR5/8-f1f	sil	2msbk	1.40	fr	cw	Few to common thin discontinuous clay films on ped faces; few fine black nodules.
Bt	36-48	7.5YR5/6 10YR5/3	7.5YR5/8-c2f&d	sicl	2msbk	1.41	fr	cw	7.5YR5/6 ped interiors; some slightly brittle areas present; common thin to moderately thick, relatively continuous clay films on ped faces; few black nodules.
2Btgb	48-69	7.5YR5/2 10YR7/1	7.5YR5/8-m2p	sicl	1cpr 1tpl 2msbk	1.52	f&slbr	cw	7.5YR5/2 ped interiors; 10YR7/1 ped faces; common moderately thick relatively continuous clay films on ped faces and few patchy thick clay films; few, fine black nodules.
2Btxgb	69-117	7.5YR5/4 10YR6/2	7.5YR5/6-c3p 7.5YR5/8-c3p 10YR6/1-f1f	sicl	1cpr 2m&csbk 1tpl	1.73	vf&br	gw	7.5YR5/4 ped interiors; 10YR6/2 ped faces; many moderately thick and thin clay films on ped faces; common black coatings on ped faces.
2BCtb	117-180	10YR5/3	10YR5/1-m3d&p 10YR5/2-m2f	sicl	1f&csbk		f&br	cs	Common to patchy thick and moderately thick clay films on ped faces; many black coatings on ped faces.
3C	180-218	10YR6/6 7.5YR6/0 7.5YR5/8 7.5YR6/8		c	m		f	gs	Subordinate colors (7.5YR6/0, 5/8, 6/8) are stratified within matrix; few thin patchy clay films on ped faces; strongly effervesces in a 10:1 dilution of concentrated HCl; carbonates are visually observed.
3R	218-239+								Rippable bedrock; bedrock is thinly bedded, interstratified limestone, calcareous shale, and calcareous sandstone.

Remarks: Field Station location - Profile description performed in a backhoe pit on Nov. 8, 1982; weather - sunny, 60-70° F; Physiography - Illinoian till plain: Parent material - Wisconsin loess over pre-Wisconsin paleosol developed in till or colluvium: Vegetation and current land use - grasses and forbs, fallow field. The fragipan in this soil is difficult to distinguish from the overlying 2Btgb horizon on the basis of color of ped faces alone. The fragipan is weakly expressed. Very coarse prismatic structure is absent or very weakly expressed. Notations from Soil Survey Manual (1951). Horizon designations from revised Chapter 4, Soil Survey Manual - Directive 430-V, Issue 1, SCS, Washington, D.C.

* f=few, c=common, m=many; 1=fine, 2=medium, 3=coarse; f=faint, d=distinct, p=prominent.

† sil=silt loam; sicl=silty clay loam; c=clay.

§ 1=weak, 2=moderate; f=fine, m=medium, c=coarse, t=thick; gr=granular, sbk=subangular blocky, abk=angular blocky, pr=prismatic, pl=platy, m=massive.

** Bulk density determined by the core method (Blake, 1965).

†† fr=friable, f=firm, br=brittle, vfr=very friable, vf=very firm, slbr=slightly brittle.

§§ c=clear, g=gradual; w=wavy, s=smooth.

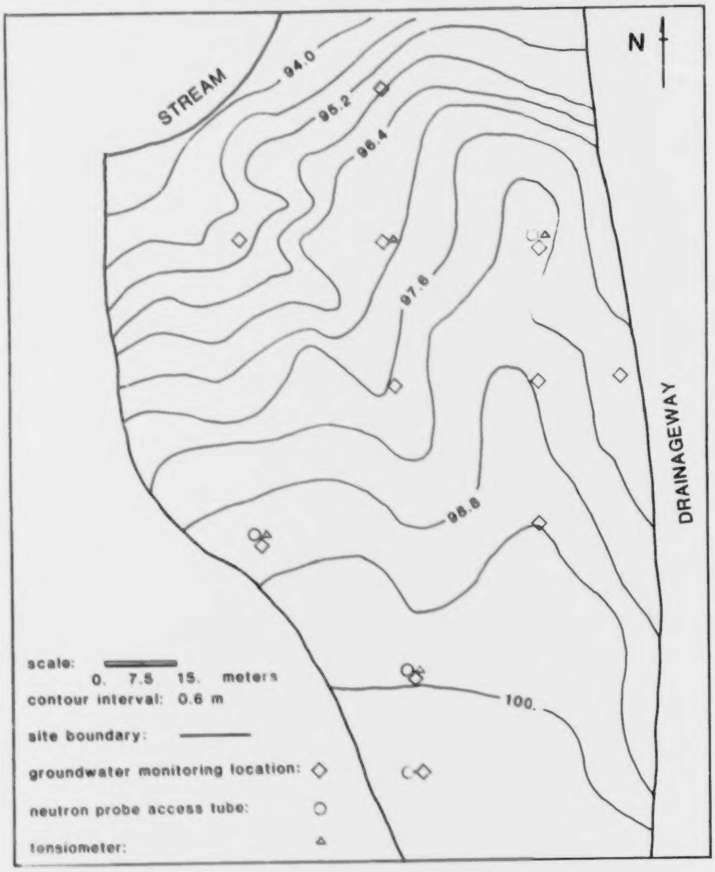

STREAM
94.0
95.2
96.4
97.6
98.8
100.
N
DRAINAGEWAY
scale:
0. 7.5 15. meters
contour interval: 0.6 m
site boundary:
groundwater monitoring location: ◇
neutron probe access tube: ○
tensiometer: △

Figure A-2. Contour map of study area.

MATERIALS AND METHODS

The study site was surveyed using standard surveying procedures. Initially, nine monitoring locations were established on a grid pattern with 30 m or 60 m between locations (Figure A-2). Two additional locations were established later in the study to obtain data in areas that required additional monitoring. Two groundwater monitoring wells were installed at each of the locations. One well was installed at approximately 1.5 m or bedrock, whichever was shallower, and the other well was installed above the fragipan or dense layer to determine if a perched water table existed.

The wells were constructed from 5-cm diameter PVC pipe that was perforated in the lower 60 cm. They were installed by hand-augering a 10-cm diameter hole to the desired depth. Five centimeters of pea gravel was placed in the bottom of the hole and the well was inserted. A gravel pack was placed in the annular space between the perforated section of pipe and the wall of the hole, and a bentonite seal was placed on top of the gravel pack. The remainder of the annular space was backfilled with soil excavated from the hole and tamped into place.

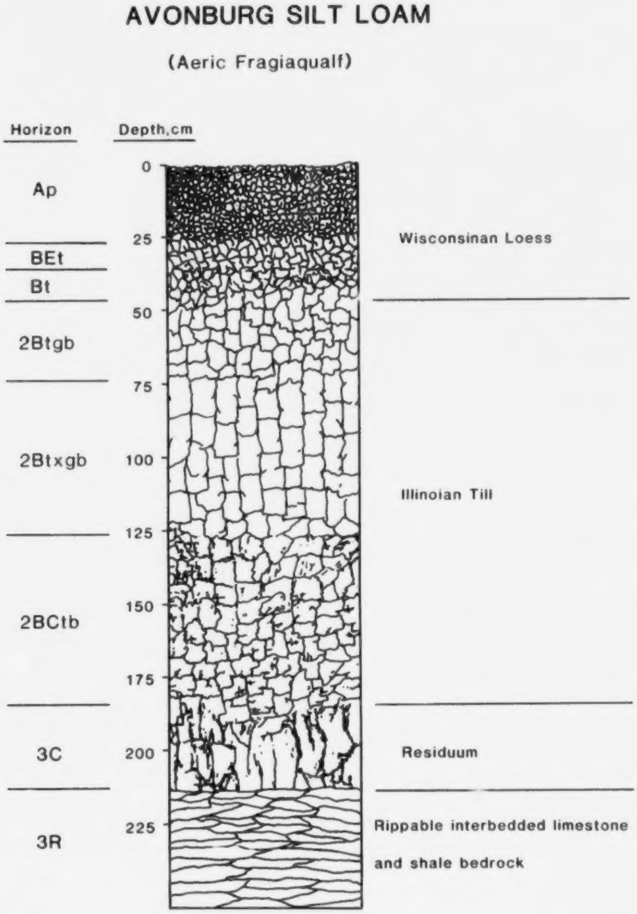

AVONBURG SILT LOAM
(Aeric Fragiaqualf)
Horizon
Depth,cm
Ap
BEt
Bt
2Btgb
2Btxgb
2BCtb
3C
3R
0
25
50
75
100
125
150
175
200
225
Wisconsinan Loess
Illinoian Till
Residuum
Rippable interbedded limestone
and shale bedrock

Figure A-3. Soil profile of an Avonburg silt loam.

Commercially available 1.9-cm diameter and 5-cm long porous cup tensiometers connected to mercury manometers were installed at four of the monitoring locations using the procedure of Bouma et al. (1974). Soil moisture contents were also measured at four of the monitoring locations with a depth moisture gauge (neutron probe) manufactured by Troxler Electronic Laboratories Inc. (Model 3222). Access tubes for the probe were constructed of thin-wall aluminum, and were installed at a depth of 1.5 m or bedrock, whichever was shallower. The installation procedure consisted of forcing the tube several centimeters into the soil and augering out the soil inside the tube. The tube was advanced into the soil in this manner until the desired depth was reached.

A tipping bucket rain gauge was used at the site to measure rainfall.

The groundwater table elevation was monitored a minimum of once per week and more often after heavy rainfall events for 2.5 yr. Measurements were made with a conductivity device that consisted of a split wire fixed to a small-diameter dowel and connected to an ohmmeter. Accuracy of 3 mm could be achieved by this method. The soil moisture content and soil moisture potential were also monitored weekly for one or more years, but the tensiometers were only read during those times of the year when they were operative, usually May through November, because tensiometers cannot be used in freezing temperatures.

The wells were occasionally tested to ensure that they were not clogged by pumping out the contents of the well and determining the rate at which the water level in the well established equilibrium with the previously measured groundwater table. The tensiometers were serviced weekly. The neutron probe was calibrated in situ for the soils in this study.

RESULTS AND DISCUSSION

Typical Groundwater Conditions

In general, the groundwater table contours followed the land surface contours over the site. Groundwater flow in the eastern quarter of the study site was in a northeasterly direction toward the drainageway that bordered the eastern side of the area. Flow in the nearly level southern and central areas was in a northerly direction, while the remainder of the flow was northwest, toward the stream tributary.

A perched water table above the fragipan was only observed on a few occasions and only at a few monitoring locations. During these periods, the free water surface in the well installed above the fragipan was higher than in the deep well. Visual observation of the soil during these periods also indicated that the soil was saturated above the fragipan but unsaturated for a distance below the fragipan. The perched condition was only present for several days.

Seasonal Variations

The monthly average groundwater table elevation for a typical monitoring location is shown in Figure A-4. The free water surface followed a sinuous yearly pattern over the site. It was generally high, within 15 to 60 cm of the soil surface, during the late fall, winter, and spring months, and was closest to the soil surface during March and April. With the increased temperatures and onset of plant growth during the late spring, the groundwater table receded, and was greater than 1.5 m, the detection limit of the monitoring wells, during the summer months. Smeck and Ransom (1983) found a similar pattern for Avonburg and Clermont soils in southwestern Ohio.

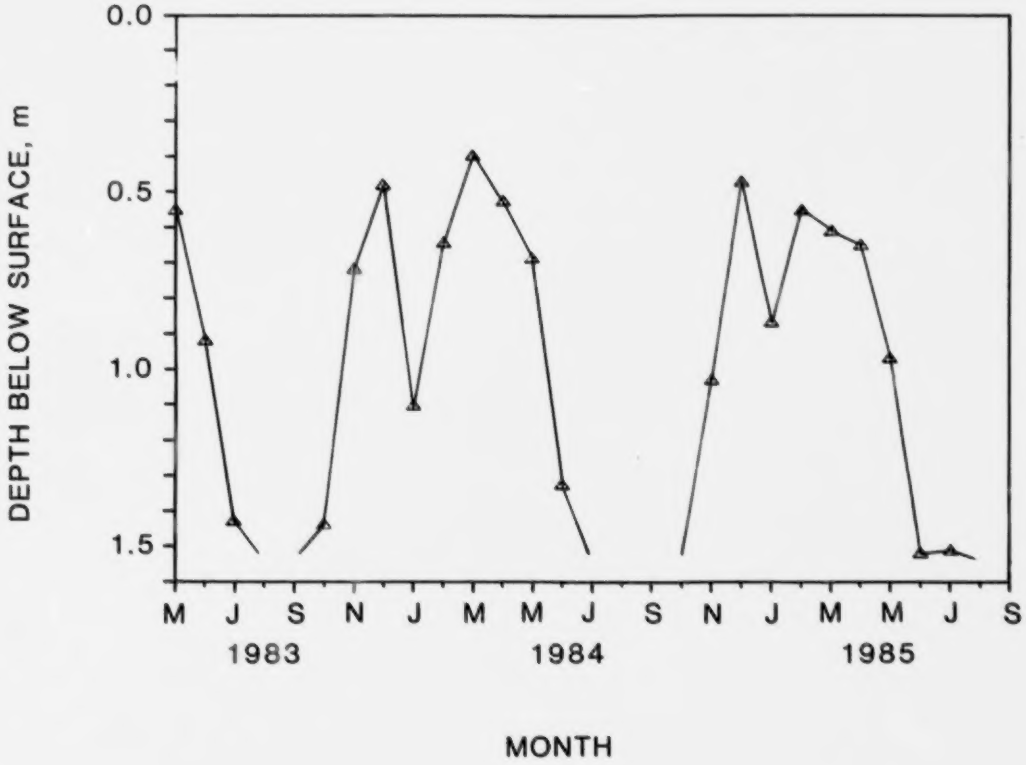


Figure A-4. Average groundwater table elevation for a typical monitoring location.

The water table also receded during January of both monitoring years. Todd (1980) reported a similar decline followed by a rise in early spring and attributed it to the presence of a frost layer above the water table. During winter, water moves upward from the water table by capillary movement and by vapor transfer to the frost layer where it freezes. Then in early spring, the frost layer begins thawing from the bottom and meltwater percolates downward to rejoin the water table.

In this study, the average temperature and rainfall during January of both years of the study were lower than normal, and physical observation showed that the frost line of the soil was at least 25 cm, and probably deeper. Therefore, the water table decline may have resulted from this upward movement from the water table to the frost line, as suggested by Todd (1980). The decline may also have resulted from the lack of groundwater recharge from surface infiltration as the result of the frost layer.

The soil moisture content, soil moisture potential, and the groundwater table depth at a typical monitoring location are shown in Figure A-5 for a time period that includes the winter through the beginning of groundwater recharge during the following fall for a year with average temperatures and rainfall. In the example, the groundwater table (Figure A-5c) was closest to the soil surface during the winter and spring, and gradually declined during the spring discharge period. It was then near or below the detectable limit of the well throughout the summer months, with an exception of the end of July, when a period of heavy rainfall resulted in a groundwater table rise. A rise was seen again in the late fall, during groundwater recharge.

The soil moisture content, presented as the percent of saturation, typically remained relatively constant during the winter and spring seasons, and decreased in the upper 1 m of the soil profile during the summer months (Figure A-5a). Because the neutron scattering method actually measures an average water content over a large sphere of influence, it must be remembered that a value presented for a particular depth, as shown in Figure A-5a, is actually the average of as much as a 15- to 60-cm sphere around the point being measured, depending on the soil moisture content at the time of measurement (Gardner, 1965).

The magnitude of soil moisture content decrease over the summer months decreased with depth. The greatest decrease occurred in the horizons closest to the soil surface (Figure A-5a, 30-cm depth), which was expected because this is the region having the most water removed by evapotranspiration. Below approximately 1 m, however, very little change in the soil moisture content occurred over most of the study area, even though the free water surface was below this depth. In the example in Figure A-5a, only approximately 7% moisture was removed from the 120-cm depth during the summer months, indicating that the soil at this depth was at or near saturation throughout the study. The soil moisture content at all depths again increased during the fall groundwater recharge period.

The soil matric potential (Figure A-5b) also suggested that the soils remained relatively wet in the lower horizons throughout the year. During the summer months the soil matric potential in the upper 1 m of the soil profile was highly negative, indicating unsaturated conditions, with the greatest matric potential occurring near the soil surface, and often exceeding -800 cm, the practical upper measurement limit of a tensiometer (Richards, 1965). The matric potential increased in the lower horizons, but only reached a maximum of about -100 cm at the 107-cm depth. The soil moisture characteristic curve of this soil showed that only 6 to 8% of the moisture of the soil was lost between a matric potential of 0 and -100 cm (Section 4 of this report). This correlates with the soil moisture content data at the 120-cm depth in this example.

These data suggest that the Illinoian till soils in this study were capable of retaining a high percentage of water even after the observed free water surface receded below the detection limits of the wells, and were saturated or nearly saturated in the lower horizons throughout most of the year.

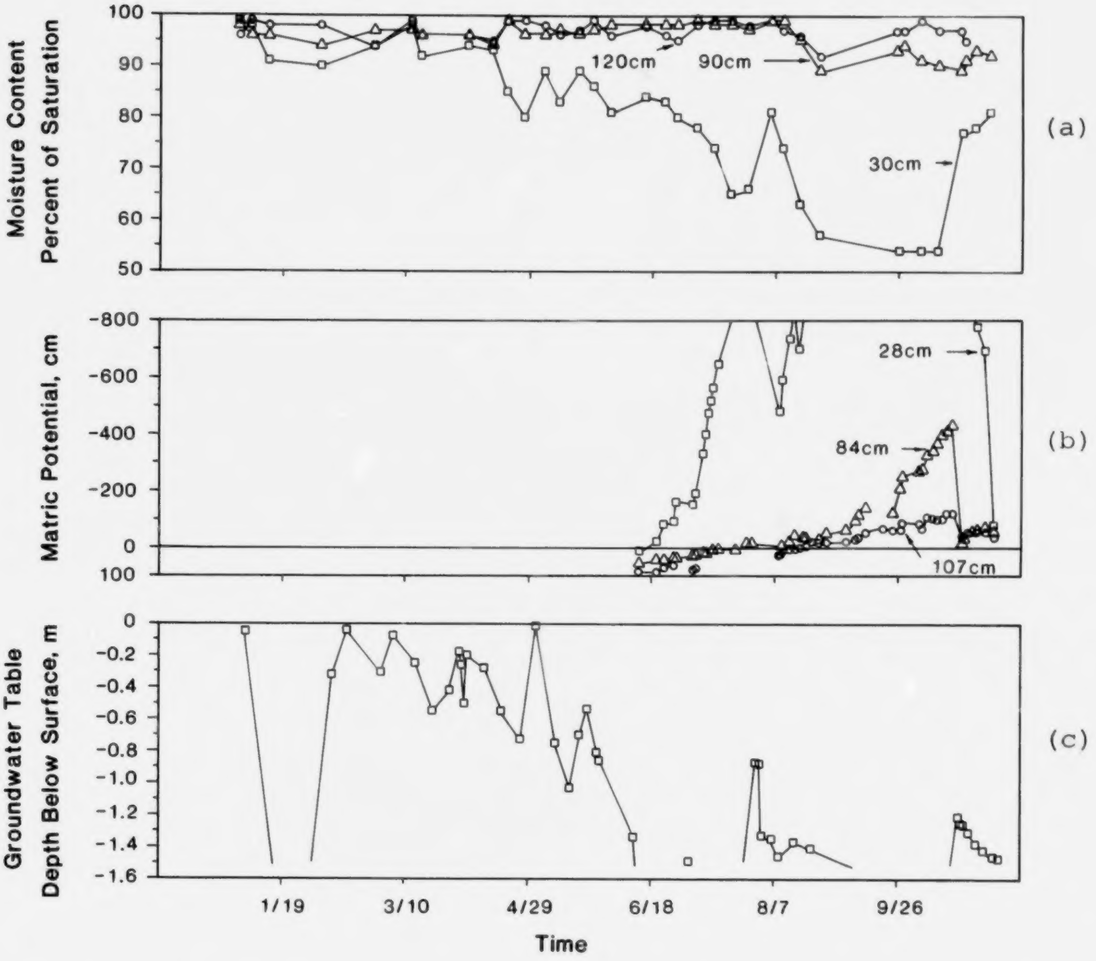


Figure A-5. Soil moisture content, soil moisture potential, and groundwater table elevation for a typical monitoring location.

Others have found little change occurring in the soil moisture content in lower horizons of soils with similar textures. Harlan and Franzmeier (1974) found that the volumetric moisture content of the 2C horizon of a Crosby silt loam soil (Aeric Ochraqualf, fine, mixed, mesic) varied only 7% over a growing season, indicating that this horizon was always close to saturation. Vepraskas et al. (1974) found that soil moisture tensions only reached 20 and 30 cm in the B31g horizon of a Grays silt loam soil (Mollic Hapludalf), which corresponded to relatively high moisture contents. They assumed that in this range, only the larger pores such as channels and planar voids were filled with air, while the fine pores in the peds remained filled with water.

An unusual occurrence during this study was that the soil moisture potentials occasionally indicated saturated conditions (positive potentials) for several weeks after the groundwater table receded below the depth of the tensiometer (Figure A-5b, 6/8 to 7/8). This usually only occurred at depths greater than 75 cm. This reaction lag may be a reverse of the type of situation that Bouma et al. (1982) described. In their study they found that large porous cup tensiometers often intersected macropores that filled rapidly when the soil surface was inundated with water, and therefore even though the soil was unsaturated, the tensiometer showed saturated conditions because it was in contact with tension-free water. In this study the soils had very large structural units, and at the deeper depths, very few macropores were observed within the structural units. Therefore, it is possible that some of the porous cups were imbedded in the micropore matrix of a structural unit, and when the macropores drained (which would occur before drainage of the micropores), the micropore matrix was not yet in equilibrium with the drained macropores, and achieving equilibrium required a longer time than expected.

Groundwater Response to Rainfall Events

Throughout the study, the observed groundwater table fluctuated continuously during the fall, winter, and spring, with the largest fluctuations occurring during the late spring and fall when the water table was receding or rising, respectively. The groundwater fluctuations observed in a representative well during a goundwater recharge period are shown in Figure A-6. In this example, the groundwater table was below the detection limit of the well until a rainfall event of 3.5 cm on October 5 resulted in a groundwater rise of at least 120 cm in one week. Because data were not collected continuously, the actual time and magnitude of rise could not be determined. After several days of no rainfall the groundwater table had receded 60 cm. Another rise of 113 cm occurred within 5 days after several large rainfall events. At this point the groundwater table was near the soil surface and remained there, with minor fluctuations, until the following summer.

The wide and rapid fluctuations observed in this study were not expected in these slowly permeable soils. However, several studies have reported that shallow water tables frequently show a disproportionate response to infiltration (Gillham, 1984). The processes that have been shown to contribute to these observed water table rises include vertical infiltration of rainfall, net lateral inflow from upslope, and air pressure buildup in the soil (Fayer and Hillel, 1982).

Vertical infiltration of rainfall can produce a groundwater table rise when rainfall percolates through the soil profile to the water table. This usually only occurs during the late fall, winter, and spring when the soil moisture content is high and little evapotranspiration occurs. The height of rise is dependent on the amount of rainfall that reaches the groundwater table and the volume of water required to fill the air-filled pores. During the summer months, most rainfall is held in the drier surface soil horizons and removed by evapotranspiration before it is redistributed to the lower soil horizons, and no groundwater recharge occurs.

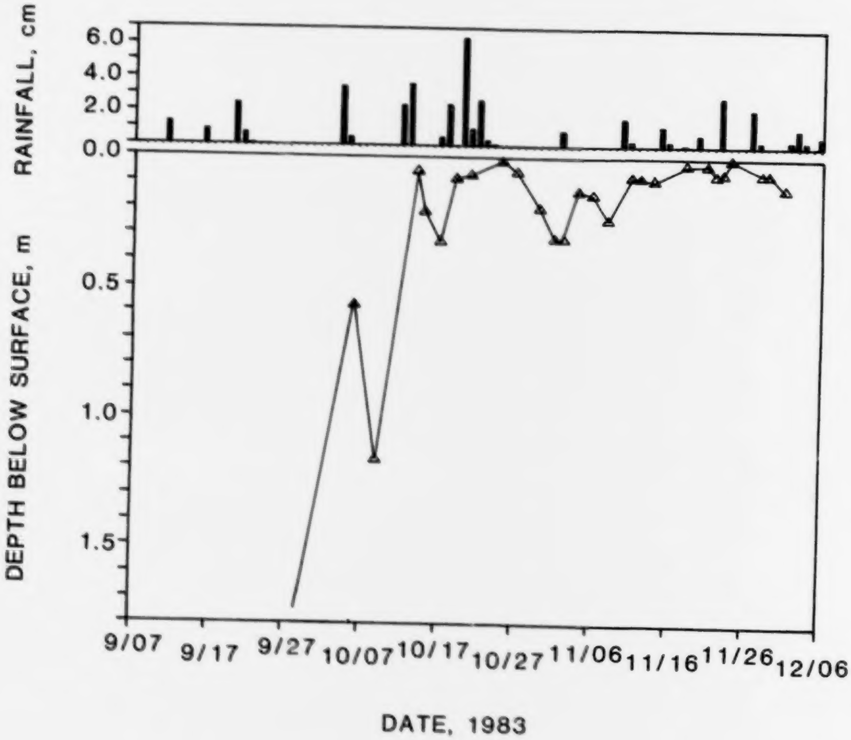


Figure A-6. Groundwater table fluctuations at a typical monitoring location.

In the soils in this study, little change was seen in the soil moisture contents in the lower horizons throughout the seasons, even when a groundwater table was not detected (Figure A-5a). This suggests that the soils had a high capillary potential, and that although the macropores drained during the summer months, little flow occurred from the micropores and they remained nearly saturated throughout the year. Therefore, during the late fall, winter, and spring, when rainfall percolated to the lower horizons, only a small volume of water was required to fill the macropores, creating a continuous horizontal free water surface. Vertical infiltration of a large rainfall event could then result in a large rise in the groundwater table.

Net lateral inflow from upslope may also have contributed to the groundwater fluctuations in this study, especially in the sideslope areas. An example of lateral flow contributing to a rise in the groundwater table is shown in Figure A-7, which shows the depth to the groundwater table in a monitoring location in the nearly level upland area, referred to as the upslope position, and a location in the sideslope area, referred to as the downslope position. During this groundwater recharge period, the groundwater table in the upslope position rose approximately 90 cm after several rainfall events, while the groundwater table in the downslope position remained below the detection limit of the well during these events. A response was not seen in the downslope position until several days later, at a time when the free water surface in the upslope position was receding. This delayed pattern occurred several times until the soils became saturated, and responses to rainfall events appeared to occur simultaneously.

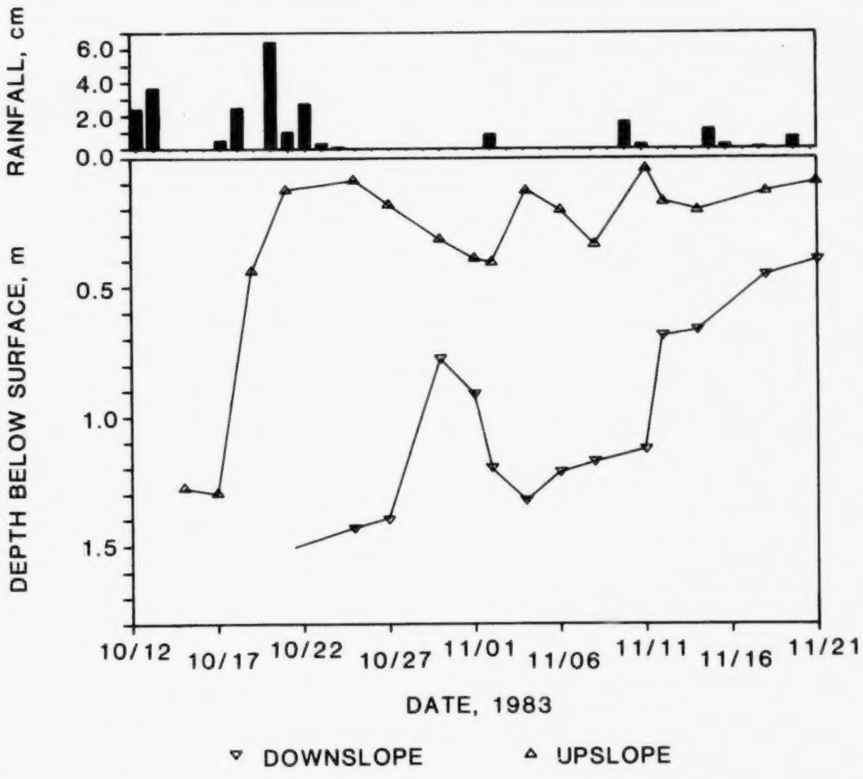


Figure A-7. Groundwater table fluctuations as a result of lateral flow.

Although other factors could have contributed to this delayed response, it is likely that lateral inflow from upslope was the dominant factor. The two sites in Figure A-7 had similar hydraulic conductivity values (Section 7 of this report) and similar soil properties, and it is therefore likely that water would percolate through the soil profiles at similar rates. However, less rainfall would infiltrate into the soil in the downslope positions because of increased runoff on the sloping sites, reducing the amount of rise attributable to vertical infiltration. Rainfall that did infiltrate into the soil could be readily removed by lateral flow over the shallow bedrock and fragipan in the sideslope area because of the larger slope gradient of these features in this area. Therefore, it is likely that no groundwater table rise occurred in the downslope areas until the soil became saturated from lateral flow from upslope.

The development of increased air pressure and/or entrapped air, another process that could have contributed to the groundwater fluctuations in the study, has been shown to occur with the combination of a high-intensity rainfall, high soil moisture content, and a horizontal air barrier such as a water table or impervious bedrock (Dixon & Linden, 1972; Fayer & Hillel, 1982; Gillham, 1984; Nielsen et al., 1959). Because there are no escape routes for air in the soil pores when there is infiltrating rainfall and a shallow water table, the air in the air-filled pores compresses and the air pressure increases above atmospheric. It is possible that infiltrating water can cause a large enough increase in soil air pressure to force water up an observation well, making it appear that there is a high water table, as shown in Figure A-8. Soil air pressures were not measured in the study. Therefore, it could not be determined whether or not air pressure buildup in the soil resulted in an observed groundwater table rise.

If air pressure buildup in the soil caused the observed fluctuations in this study, then no actual fluctuation occurred in the groundwater table. However, the soil moisture content data and the variation in the time of groundwater rise over the site strongly suggest that both vertical infiltration of rainfall and net lateral flow from upslope could also contribute to a rapid and large rise in the actual groundwater table. Therefore, it is likely that all of these processes contributed to some degree to the wide fluctuations in the observed groundwater table. The actual fluctuations may not have been as large as what was observed because of air pressure buildup, but it is likely that some degree of fluctuation was occurring, and that the groundwater table was almost never stable.

Landscape Position Effects

The water table level in the soils in the nearly level upslope positions generally fluctuated with a greater magnitude and frequency than in the soils in the sloping sideslope positions. These varied responses may have resulted from the various processes that contributed to the observed groundwater table rise. In the nearly level upslope positions, vertical infiltration of rainfall and air pressure buildup in the soil were probably the main mechanisms by which an observed rise occurred. Because little runoff occurs in a nearly level landscape position, most of the rainfall that falls on the soil surface can infiltrate and percolate through the soil profile. Not only can more water reach the water table, but a more continuous wetting front can be created by the infiltrating rainfall, resulting in air pressure buildup in the soil, producing a larger observed fluctuation than actually occurs. In the sloping sideslope areas, net lateral inflow from upslope is most likely the main contributor to an observed groundwater rise, as previously mentioned.

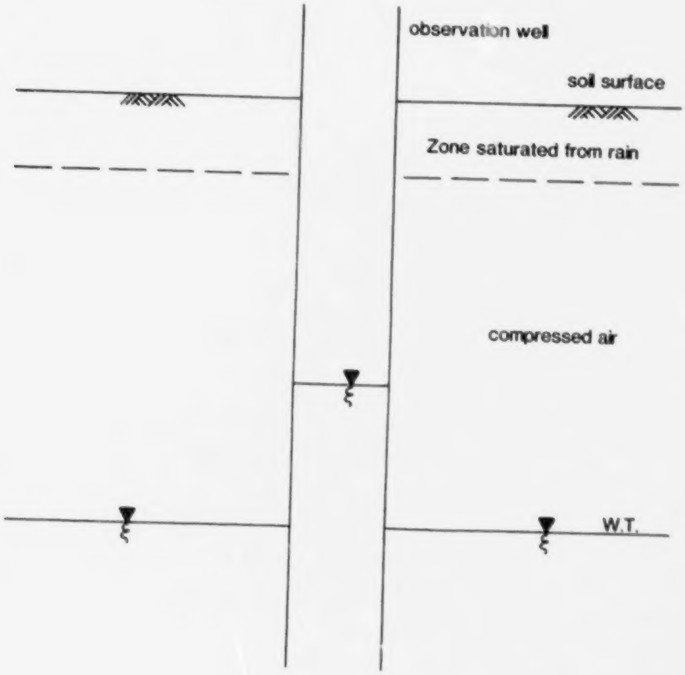


Figure A-8. Groundwater table rise in an observation well resulting from air pressure buildup in the soil (from Todd, 1980. Fig. 6.22, p. 241).

Because of these various methods of groundwater recharge, it is likely that the soils in the downslope positions became saturated mainly by lateral inflow from upslope, while the soils in the upslope areas became saturated by vertical infiltration of rainfall. This method of saturation, among other factors, appeared to affect the length of time the soils were saturated. During years with typical weather patterns, the groundwater table in the nearly level upslope landscape positions, which were typically Avonburg soils, remained nearest the soil surface longer in the spring and rose to high levels sooner in the fall than the soils in the gently sloping to sloping downslope positions, which were predominantly Rossmoyne soils, the moderately well-drained soil of the catena. Figure A-9 shows an example of this during the fall of 1984. Locations 1 and 15 represent the nearly level upslope positions and locations 5 and 11 represent the sloping downslope positions. In this example, the time of groundwater rise correlated with the slope position, the rise occurring first in the most upslope position and last in the most downslope position. Franzmeier et al. (1983) also found that the rise and fall of the water table level varied among soils. For several of the soil catenas in their study, the water table curve for the somewhat poorly drained soils lagged about 1 month behind the curve for the well- or moderately well-drained soils in the spring and early summer.

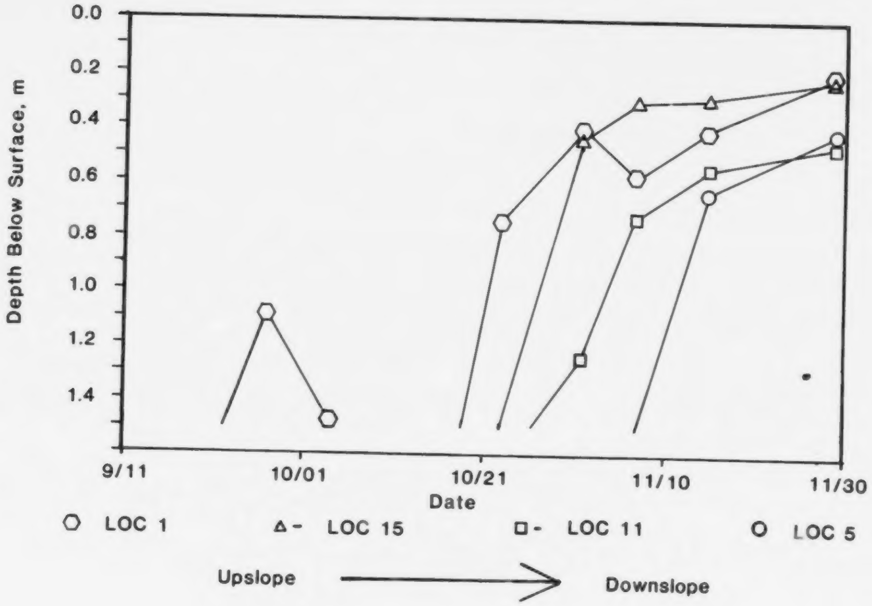


Figure A-9. Variation in time of groundwater recharge resulting from landscape position.

The variation in the time of groundwater rise or fall is most likely the result of soil moisture content differences that are due to landscape position. The soils in the nearly level upslope positions can remain saturated longer in the spring and can become saturated sooner in the fall because little runoff occurs in these landscape positions, and therefore most of the rainfall that falls on these areas can infiltrate and percolate through the soil. Also, less lateral flow would occur within the soil profile because of the low slope gradient.

In the sloping, downslope positions, precipitation can run off the site and less water infiltrates into the soil. Therefore, the soils in these areas will not become saturated as soon as those in the upslope positions. Also, more water could be removed internally from these soils by lateral flow as a result of the steeper slope gradient at these sites.

Soil Morphological Indications of Wetness

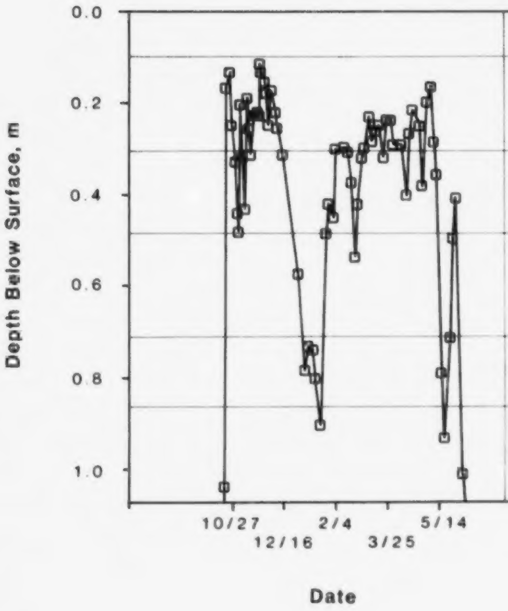
The soil mottling patterns and corresponding groundwater table elevations are presented for a typical monitoring location in Figure A-10. In the example, the Ap2 horizon (10 to 30 cm) contained manganese (Mn) nodules and faint 3-chroma silt cutans (coatings). The groundwater table was observed within this horizon 33% of the time over two years of data collection. In the BE horizon (30 to 48 cm), Mn nodules and cutans were present along with more distinctive 3-chroma silt coatings. The groundwater table was observed within this horizon 47% of the time. Although the groundwater table was observed within these two horizons for extended periods of time, they did not contain mottles with chromas of 2 or less, those frequently used to indicate that the soil horizon is saturated with water at some period of the year (Soil Survey Staff, 1975). The presence of 3-chroma mottles and Mn nodules and cutans in horizons that do not contain mottles with chromas of 2 or less but are saturated for short periods has been observed in other studies (Daniels et al., 1971; Franzmeier et al., 1983; Vepraskas & Wilding, 1983).

Mottles with chromas of 2 were first present in the 2Btgb horizon, at a depth of 48 cm. The groundwater table was observed within this horizon 51% of the time. The 2-chroma mottles were in the form of clay coatings and as mottles within peds. Ferrans (iron cutans) were also present in this horizon.

The lack of mottles with chromas of 2 or less in horizons that are saturated with water may be the result of periods of saturation that are not associated with periods of reduction. Although the soil pores are filled with water, enough dissolved oxygen may still be present in the water to maintain aerobic conditions, or, because dissolved oxygen must first be reduced before the reduction of Fe(III) proceeds, to keep the process of reduction from proceeding to the extent of reducing Fe(III), a condition necessary for mottle formation (Vepraskas & Wilding, 1983).

Vepraskas and Wilding (1983) found that periods of saturation and reduction did not coincide in several Alfisols in Texas. Smeck and Ransom (1983) also found that the unsaturated zone and upper section of the saturated zone in soils similar to the soils in this study were nonequilibrium systems. This finding was supported by the simultaneous occurrence of dissolved oxygen with reduced iron and a mixed redox potential that did not correlate with dissolved oxygen content. Daniels et al. (1971) found that the dissolved oxygen content of water from the zone of saturation in soils on the coastal plain of North Carolina could remain high enough to prevent iron reduction even when the water table was near the soil surface for four or five months each year.

In the example shown in Figure A-10, while the Ap2 and BE horizons were saturated for a large percentage of time, the continuous length of saturation was short because the groundwater table constantly fluctuated through these horizons. Oxygen could constantly be introduced into the horizons, preventing the formation of reducing conditions. However, in the 2Btgb horizon, few fluctuations occurred and the groundwater table was within this horizon for extended periods, allowing the formation of reducing conditions.



Horizon	% of Time Groundwater Observed Within Horizon	Soil Morphological Features
Ap1	0	
Ap2	33	Mn nodules 3-chroma cutans
BE	47	Mn nodules & cutans 3-chroma cutans
2Btgb	51	Mn nodules & cutans Fe cutans 3-chroma cutans 2-chroma mottles & cutans
2Btxgb1	55	Mn nodules & cutans 1-chroma mottles & cutans
2Btxgb2	60	Mn nodules & cutans Fe cutans 2-chroma mottles & cutans

Figure A-10. Depth of the observed groundwater table and corresponding soil morphological features for a typical monitoring location in an Avonburg silt loam soil.

It is also possible that air pressure buildup in the soil resulted in the observed groundwater fluctuations, so that the Ap2 and BE horizons were not actually saturated. But as previously mentioned, although air pressure buildup probably occurred, the groundwater table still appeared to be affected by vertical infiltration of rainfall and by lateral inflow from upslope, so it is likely that some degree of groundwater table fluctuation occurred. Also, the 3-chroma mottles and Mn nodules present in these horizons suggest that some period of saturation occurred.

It is apparent from this and other studies that mottles with chromas of 2 or less are not always formed during saturation, and that in some soils more information should be obtained to describe the soil moisture regime accurately. As a result of previous studies, several proposals have been developed to revise the current criteria used to recognize soil moisture regimes. Vepraskas and Wilding (1983) proposed to include those conditions with dominant chromas of 3 on ped exteriors when accompanied by a mottled ped interior with matrix chromas of 4 or more in the aquic moisture regime. Franzmeier et al. (1983) suggested three classes of soil wetness and aeration as related to soil color pattern, including 2-chroma colors to indicate periodic saturation, and 3-chroma colors to also indicate periods of saturation, but less reduced than horizons with 2-chroma colors. Veneman et al. (1976) formed three broad categories of soil moisture regimes and associated morphological features using Brewer's terminology (Brewer, 1964) to expand mottle descriptions. One category included mottling dominated by high chromas in peds, ped mangans and few Fe mottles, and was associated with very short periods of saturation.

The results of this study support the suggestions that additional categories are required to accurately describe the soil moisture regime for the soils in this study.

SUMMARY AND CONCLUSIONS

Soil moisture characteristics of a 1.2-hectare Illinoian till plain site were monitored weekly for 2.5 yr to determine seasonal and topographical variations over the small land area, and to determine whether soil morphological features could accurately predict these variations.

In general, there was great variability in the groundwater table elevation and its response to climatic events over the small area of land. The free water surface was within 15 to 60 cm of the soil surface during the late fall, winter, and spring. During the summer months it receded to 150 cm or deeper over most of the study area. Large fluctuations occurred in the free water surface during both the rise and fall of the groundwater table, when soil moisture contents were high. These fluctuations were most likely the result of vertical infiltration of rainfall or of lateral inflow from upslope.

Landscape position appeared to influence the time and magnitude of the groundwater table rise and fall. The groundwater table in the nearly level upslope positions receded later in the spring and rose sooner in the fall than in the gently sloping downslope positions. Groundwater response to rainfall events generally appeared first in the nearly level upslope positions.

The soil moisture content changes over the seasons correlated with the groundwater table changes in the surface 1 m of the soil. However, below this depth, very little change occurred in the soil moisture content, indicating that the soil retained much moisture even when the groundwater table was below the area measured. This could have important implications for land use, if one assumes that the soils are relatively dry just because a water table is not present. For uses such as onsite wastewater disposal, this near-saturated condition can be just as prohibitive as when a water table is present.

Soil morphological features, including nodules, cutans, and mottles could be used to predict the presence and duration of the high water table in these soils. This study agrees with the proposals to include additional morphological features in interpreting the high water table.

This study showed that occasional monitoring of groundwater wells in a few landscape positions may not be sufficient to adequately describe groundwater patterns. Additional monitorings of the soil moisture content and soil moisture potential may also be required to describe the soil moisture regime when a groundwater table is not present.

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Appendix B

ALTERNATING SOIL ABSORPTION FIELD SYSTEM

Margaret Cashell David Effert

INTRODUCTION

Conventional soil absorption systems have been used successfully for many years to treat and dispose of domestic wastes. Properly designed, installed, and maintained systems usually function adequately for approximately 20 years (Sharpe et al., 1984), but often ultimately fail as the result of a natural accumulation of organic and inorganic materials at the soil-gravel interface. This accumulation, or clogging mat, can clog the soil to a point where insufficient amounts of septic tank effluent can pass through (USEPA, 1980), leading to ponding of sewage above the absorption field or surface breakthrough downgrade from the system.

Solutions for a failed absorption system have generally been costly. The most common techniques are to replace the failed system with a new one or to rebuild the existing system after removing the clogging layer (Bishop and Logsdon, 1981). An effective method for rejuvenating a failed system so that replacement is not needed is to take it out of service for a prolonged period of time and rest it (USEPA, 1980). With resting, the field gradually drains and the clogged infiltrative surface is exposed to air, creating aerobic conditions. This hastens decomposition of the clogging materials, restoring much of the infiltrative capacity of the soil (Winneberger et al., 1960).

Rejuvenation can be accomplished by alternating effluent disposal between two soil absorption fields. One field is in use while the other field is "rested," allowing restoration of the infiltrative capacity of the field.

An alternating soil absorption field system was installed at a private residence in Hamilton County, Ohio, where an existing conventional system had failed after 18 years of satisfactory operation. No outlet was available at the site for a surface water discharge system, and the area available for a replacement system was small. Therefore, a reasonable option to correct the failure situation at the site was to install additional soil absorption trench in a second area.

The amount of additional trench that was installed was similar to the amount in the existing system which had apparently been satisfactory for 18 years even though it was grossly undersized for the 3-bedroom home. Because the amount of absorption area in the two systems were of similar size, effluent was alternated between them in an attempt to restore the infiltrative surface in the existing system.

The primary objective of this study was to correct a failure situation at a private residence. A secondary objective was to determine if the failed existing system could be rehabilitated through resting, and to determine if the alternating soil absorption field concept is an effective type of alternative system in southwestern Ohio.

MATERIAL AND METHODS

The site in this study was located in western Hamilton County, Ohio. The original residents at the location, four adults and one child at the time of this study, occupied the 3-bedroom home. The daily wastewater applied to the soil absorption system averaged 700 lpd.

The existing system consisted of a 2.84 m^3 concrete septic tank and 38 m of lateral, spaced into three trenches that were installed across a 10% slope (Figure B-1). The trenches were installed at an average depth of 75 cm, and had an average height of 35 cm of gravel. In the northern section of the system, bedrock occurred within 80 cm, indicating that only several centimeters of soil separated the trench bottom from the bedrock. Bedrock was deeper than 110 cm in the southern section.

An onsite investigation was performed to determine if the soils and site at this location were suitable for the installation of a second system. The area available for an additional system was a 7%, north-facing sideslope. The morphological properties of a representative soil profile at this location are given in Table B-1. Fill material covered the original soil to a depth ranging from 0 to 1 m, and had been in place for 20 years. The fill was composed largely of clay and clay loam textured residual soil, and was stratified, suggesting that the fill had been placed in layers. Soil mottles (spots of contrasting color that can indicate the presence of a fluctuating high water table) were present throughout the fill with the exception of one layer of variable thickness that had a silt loam texture. Large flat rocks, most likely from the underlying interbedded limestone and shale bedrock, were found in the fill.

The original topsoil at this site, buried by the fill material, consisted of 20 cm of dark grayish brown silt loam. Strong brown mottles were present in the upper portions of the topsoil horizons, but were not present in the lower zones. This mottle pattern indicated that the textural discontinuity between the fill material and the topsoil impeded vertical water movement through the profile, resulting in a zone of fluctuating saturation at the soil-fill interface.

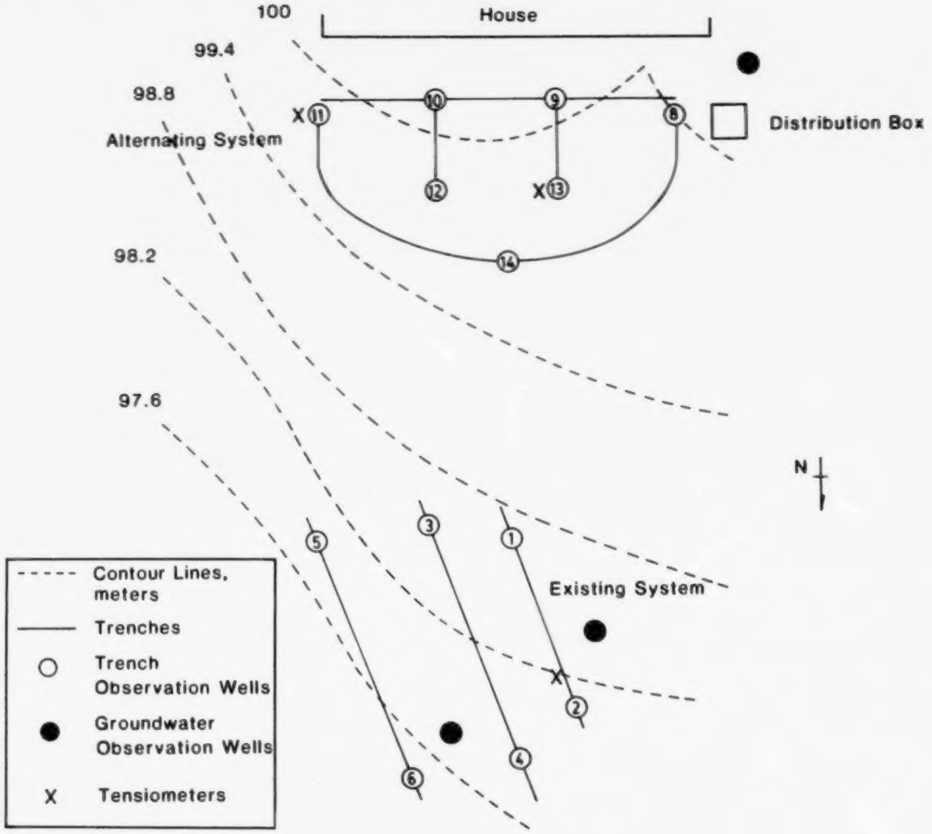

100
House
99.4
98.8
Alternating System
X 11
10
9
8
Distribution Box
12
X 13
14
98.2
97.6
N
5
3
1
Existing System
X
2
4
6
- - - - Contour Lines, meters
—— Trenches
○ Trench Observation Wells
● Groundwater Observation Wells
X Tensiometers

Figure B-1. Location of the existing system, alternating system, and the monitoring devices.

The subsoil at this site was approximately 30 cm of dark yellowish brown silty clay loam material. This material was mottled throughout, especially in the lower part of the horizon. A mottled, yellowish brown clay loam soil occurred at a depth of 94 to 109 cm. Interbedded limestone and shale bedrock was found at about 180 cm.

Six percolation tests were performed in the area where the alternating system was to be installed. The percolation rates varied considerably because of the fill material that was present, ranging from 3 min/cm to 32 min/cm.

TABLE B-1. MORPHOLOGICAL PROPERTIES OF SOIL AT ALTERNATING FIELD SITE

Horizon	Depth, cm	Munsell Color, moist matrix	mottles*	Hand texture†	Structure§	Roots**	Boundary††	Consistency§§	Remarks
C	0-41								Fill material composed largely of c and cl residual soil. Few channers and flagstones present.
2Ab1	41-52	10YR4/2	7.5YR4/6-m1d	sil	2csbk	1f	cw	fr	
2Ab2	52-61	10YR4/2 10YR4/4		sil	2msbk	1f	cw	fr	Matrix color is 10YR4/4 rubbed.
2Btb1	61-72	10YR4/4	2.5YR5/4-f1f	sicl	2msbk	none	cw	fr	
2Btb2	72-86	10YR4/4	2.5YR5/4-m1f 5YR/6/2-c2d	sicl	2msbk	none	cw	fr	
2Btb3	86-94	10YR5/6 5YR6/2 7.5YR5/6 2.5YR5/2		sicl	2msbk	none	d		Matrix colors are mixed. Rubbed color is 10YR4/4.
3C	94-109	10YR5/6	2.5YR6/2-m3d	cl	m	none			
3C	109+	10YR5/6	2.5YR6/2-m3d 10YR2/2-m1p	cl	m	none			10YR2/2 color are concretions.

Remarks: Robert Fischesser location; Profile description performed on 9/16/82 in a trench; Landscape position and slope - side slope, 7% - north facing; parent material - fill over till over weathered limestone/shale bedrock; Vegetation and current land use - bluegrass, lawn.

* f = few, c = common, m = many; 1 = fine, 2 = medium, 3 = coarse; f = faint, d = distinct, p = prominent.

† sil - silt loam, sicl - silty clay loam, cl - clay loam.

§ 2 = moderate; m = medium; c = coarse; sbk - subangular blocky; m - massive.

** 1 = few; f = fine.

†† c = clear, d = diffuse, w = wavy.

§§ fr = friable.

Fill material of the nature found at this site is not usually considered to be suitable for soil absorption fields. However, the material had been in place for 20 years, and it was felt that it had stabilized and would not change further in the near future. Also, there was undisturbed soil material below the fill that appeared to be unsaturated to provide treatment and disposal. As previously mentioned, onsite discharge was the only option at this site because there was no outlet for a surface-water discharge system, and there was not enough land area for a mound system. Therefore, it was felt that installing additional soil absorption trenches in the fill material was the most reasonable option at this site.

Before installation of the new system, the existing septic tank was pumped out, inspected, and the baffles were replaced. Because of the small lot size, only an additional 32 m of trench could be installed at the site. A minimum of 20 cm of washed gravel was placed below the 10-cm diameter distribution pipe and at least 5 cm of gravel covered the pipe. A synthetic fabric was used to separate the backfill from the aggregate. A distribution box was used to distribute the effluent. The location of the alternating system is shown in Figure B-1.

Tensiometers and observation wells were installed in both the existing and alternating systems to monitor the soil moisture potential in the soil surrounding the trenches and the ponding depths in the trenches, respectively. The commercially available tensiometers consisted of 1.9-cm diameter and 5-cm long porous cups connected to mercury manometers. In the alternating system, two sets of tensiometers were installed, one in the upper trench and one in the lower trench. The location of these sets is shown in Figure B-1. One tensiometer in each set was located directly below the trench so that effluent movement out of the trench bottom could be observed. Three tensiometers in each set were installed in the sidewall at various heights above the trench bottom to observe the flow of effluent through the sidewall (Figure B-2). One tensiometer in each set was located 1.5 m away from the trench to obtain the soil moisture potential in an area less influenced by the trench.

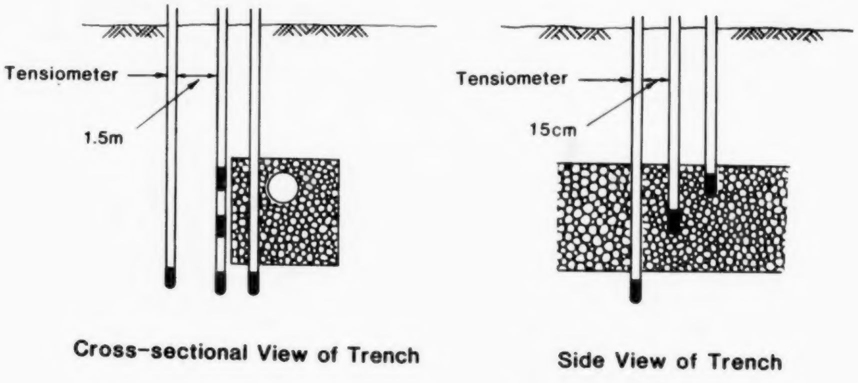


Figure B-2. Location of tensiometers in alternating trenches.

In the existing system, one set of tensiometers was installed in the trench that was at the upslope position, as shown in Figure B-1. Because this system was installed very close to the bedrock, tensiometers could not be installed below the trench, and the soil moisture potential below the trench was not monitored. The three tensiometers in this set were installed in the sidewall area.

The tensiometers in both systems, with the exception of the tensiometers in the trench bottoms in the alternating system, were installed in a manner similar to that described by Bouma et al. (1974). To install the tensiometers in the trench bottoms, a hole with a slightly smaller diameter than the porous cup was first made in the soil at the bottom of the observation well. The tensiometer was then pushed into this hole, and soil was pressed against the plastic shaft connected to the porous cup to prevent water from infiltrating into the cup area. The space surrounding the shaft of the tensiometer was not backfilled, as is usually suggested (Bouma et al., 1974), because the shaft was in the trench itself.

The observation wells were installed at the locations shown in Figure B-1. The wells were 5-cm diameter plastic pipe that was perforated in the lower section to maintain hydraulic continuity with the water in the trenches. The observation wells in the alternating system were placed in the trenches during construction. In the existing system, the wells were installed by hand-augering to the infiltrative surface, taking care not to disturb the mature clogging mat. Three additional wells were installed outside of the trenches, on top of the bedrock, to determine if a fluctuating high groundwater table occurred above the bedrock at this site.

Ponding depth measurements were made at least once per week during the three years of the study. Because tensiometers cannot be used in freezing temperatures, the tensiometers were read weekly only during those times of the year when they were operative, usually May to November. Rainfall measurements were made weekly using a tipping bucket rain gauge.

It was initally planned that the existing system at this site would be kept out of service for at least one year to restore the infiltrative surface. The two systems would then alternate in service for 6-month intervals. Unforeseen problems occurred, however, and system alternation was performed as shown in Table B-2. The data are presented in terms of these phases of operation.

RESULTS AND DISCUSSION

System Operation--Phase I

The alternating system was installed in the fall of 1982. Wastewater was not immediately diverted to this system in order to establish baseline ponding depths in the existing system. This baseline data collection period comprises Phase I. Ponding depths for representative wells in the existing and alternating systems are presented in Figure B-3. When the alternating system was first installed, the ponding depths in the existing system decreased (Figure B-3, Well 4). It was found that the distribution box had cracked, allowing effluent to seep from it, and thus the entire volume of effluent was not entering the existing system. This, and the fact that the autumn of this year was relatively dry, resulted in the decrease in ponding depths in the existing system. When the problem was detected, the distribution box was fixed and the ponding depths in the existing system increased, and surfacing of the effluent again occurred, signifying failure.

TABLE B-2. OPERATION SCHEDULE FOR THE ALTERNATING SOIL ABSORPTION FIELD SYSTEM

Phase of operation	Date	Days in service*	System in operation
I	9/16/82-5/26/83	1-253	Existing system
II	5/26/83-9/27/83	253-377	Alternating system
	9/27/83-11/16/83	377-427	Alternating system-greater volume of effluent directed to upper trench
III	11/16/83-2/20/84	427-523	Existing system
IV	2/20/84-6/13/84	523-637	Alternating system
	6/13/84-7/20/84	637-674	Alternating system-lower trench only
	7/20/84-12/21/84	674-828	Alternating system-lower and upper trenches
V	12/21/84-9/3/85	828-1084	Existing and alternating systems

* The days that the systems were in service beginning with the time that monitoring was initiated.

Before effluent had been diverted to the alternating system, some ponding was evident in the lower trench of this system (Figure B-3, Well 11). It was not determined whether the liquid was groundwater or septic tank effluent. Because of the cracked distribution box, however, it is likely that effluent had entered this system.

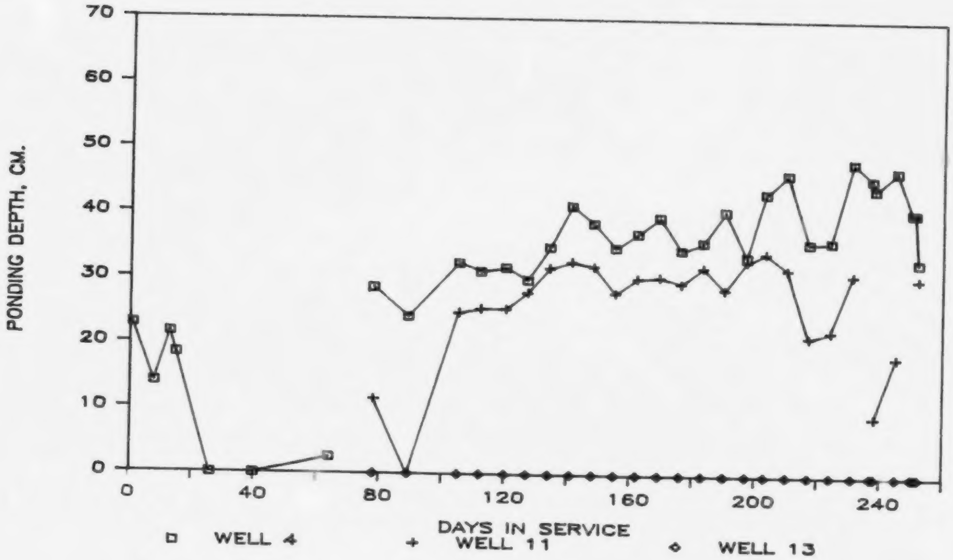


Figure B-3. Ponding depths in representative wells in the existing and alternating systems for Phase I (9/16/82-5/25/83).

Phase II

Effluent was diverted to the alternating system in May 1983 (Phase II). After this, the ponding depth in the existing system gradually decreased and eventually dried out after approximately 2 months of nonuse (Figure B-4, Well 4). Water again ponded in this system in the fall, but was the result of a high groundwater table at the site.

The ponding depth in the trenches in the alternating system gradually increased after the effluent was directed to them (Figure B-4, Phase II). A distribution problem was found between the upper and lower trench of this system, however, and the upper trench was receiving a lesser volume of effluent than the lower trench. Thus, the ponding depth in the upper trench (Figure B-4, Well 13) did not have the same magnitude of increase as the lower trench (Figure B-4, Well 11). Also, the lower trench had been inundated with liquid since the system was installed and it was likely that some clogging of the infiltrative surface occurred before the system was actually put into operation, causing greater ponding. Even when a greater volume of effluent was applied to the upper trench (days 377 - 427), the ponding depth remained higher in the lower trench.

The tensiometer data obtained from the existing system during Phase II is shown in Figure B-5. The depths shown indicate the height of the tensiometer above the trench bottom. The soil moisture matric potential in the trench sidewall became negative (indicating unsaturated soil conditions) 2 months after the effluent was directed to the alternating system. This closely corresponds to the length of time required for the trenches to dry out. As the high summer evapotranspiration rate and temperatures decreased during the fall, the soil moisture content increased, resulting in less negative matric potentials, until October, when saturated conditions again occurred as the trenches were inundated with groundwater.

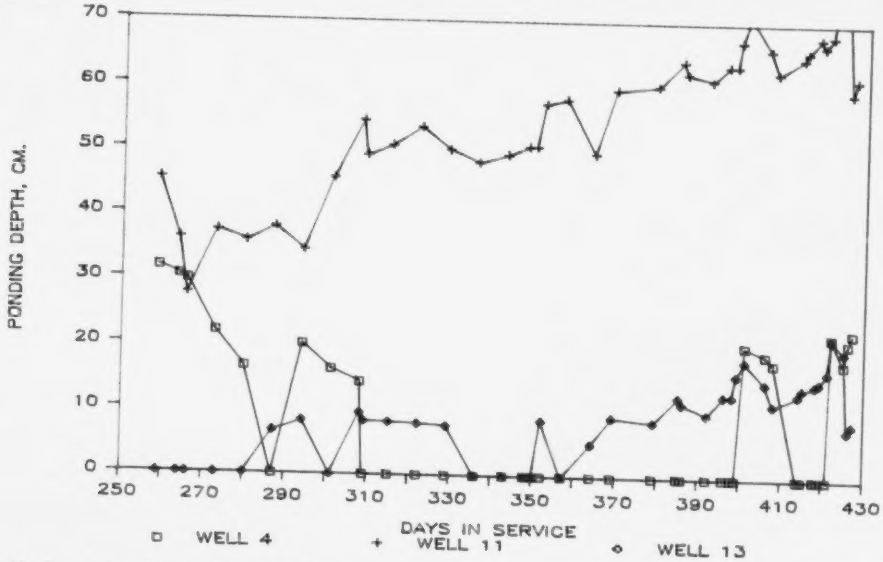


Figure B-4. Ponding depths in representative wells in the existing and alternating systems for Phase II (5/26/83-11/16/83).

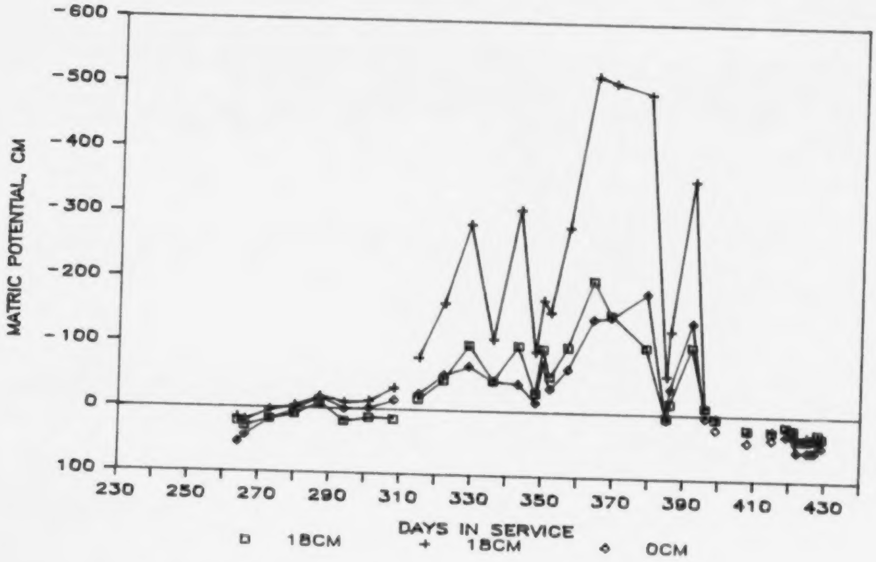


Figure B-5. The matric potential in the sidewalls of the existing system during Phase II (4/23/83-11/29/83).

The tensiometer data presented for the alternating system include that for the tensiometer installed in the trench bottom and for two of the sidewall tensiometers in both the upper and the lower trench. The soil matric potentials for the lower trench of the alternating system during Phase II are shown in Figure B-6. The soil below the trench was saturated (positive potential) throughout Phase II, suggesting that saturated flow was occurring out of the bottom of the trench. The sidewall tensiometers showed that the soil was saturated at the bottom elevation of the trench (0 cm) throughout Phase II, but was mostly unsaturated 30 cm above the trench bottom, even when the ponding depth was above this depth. This suggests that the greatest volume of the flow occurred out of the bottom of this trench during Phase II, since saturated flow is more rapid than unsaturated flow.

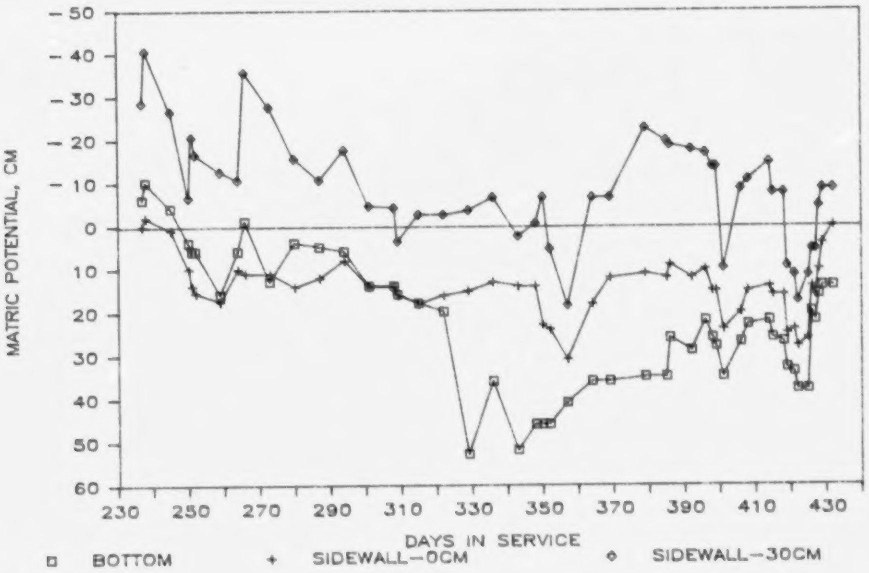


Figure B-6. The matric potential below the trench and in the trench sidewalls in the lower trench of the alternating system during Phase II (4/23/83-11/29/83).

The tensiometer data for the upper trench in the alternating system are presented in Figure B-7. The tensiometer directly below the trench showed slightly positive potentials (saturated conditions) until the end of August. Unsaturated conditions then occurred until the middle of October, during which time the ponding depth in the trench was very low (Figure B-4, Well 13). The tensiometers located in the sidewall of the upper trench showed that mostly unsaturated soil conditions were present throughout Phase II. With the low ponding depth of effluent in this trench, this was expected.

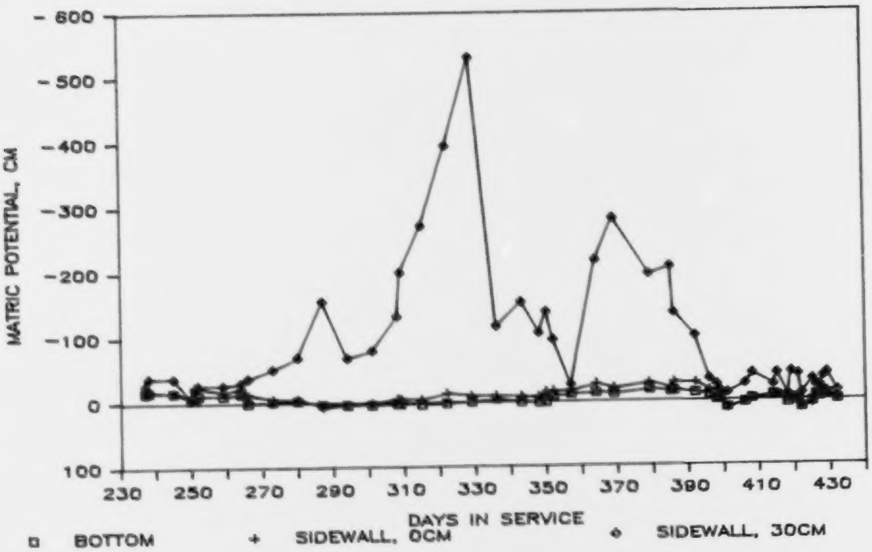


Figure B-7. The matric potential below the trench and in the trench sidewalls in the upper trench of the alternating system during Phase II (4/23/83-11/29/83).

During Phase II, the control tensiometers showed greater negative (more unsaturated) matric potentials over the summer months and were greater than the upper measurement limit of a tensiometer, -800 cm (Richards, 1965), during most of the phase.

Phase III

Wastewater was directed to the existing system during November 1983 to fix the distribution problem between the upper and lower trenches in the alternating system that was present during Phase II. The ponding depths during Phase III are shown in Figure B-8. The ponding depth in the existing system (Figure B-8, Well 4) was already high as the result of groundwater intrusion, and little change in the ponding depth occurred with the addition of effluent. The trenches in the alternating system quickly dried out and remained dry during this time period (Figure B-8, Wells 11, 13).

Phase IV

The effluent was diverted back to the alternating system in February 1984. The ponding depths in the existing system remained high until spring and then slowly decreased through June, when water was no longer ponded in the trenches (Figure B-9, Well 4). The trenches remained dry for approximately 60 days, and as in the previous year, were again inundated with groundwater during the fall.

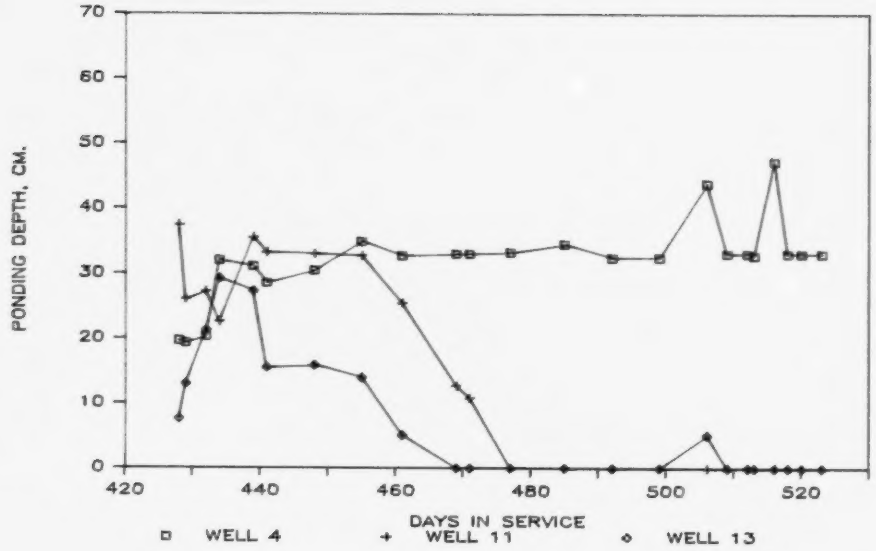
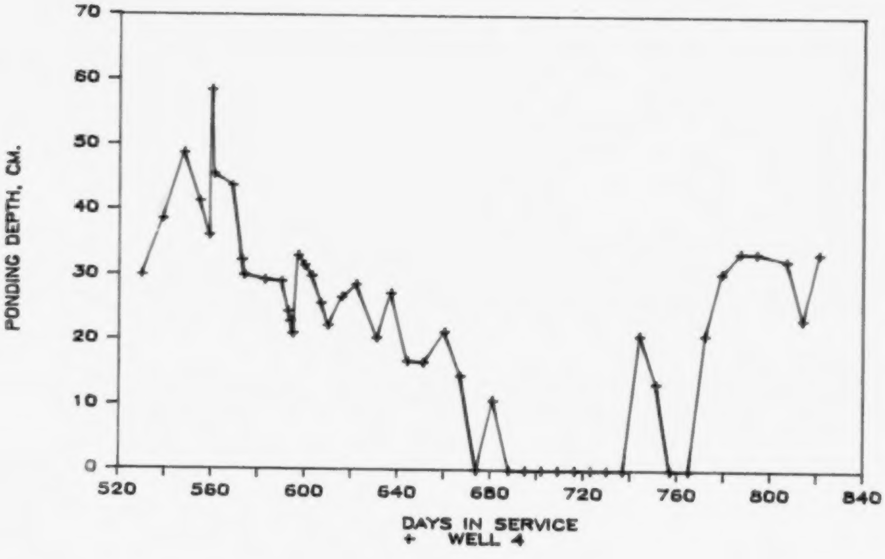


Figure B-8. Ponding depths in representative wells in the existing and alternating systems for Phase III (11/17/83-2/20/84).

The ponding depth in the alternating system quickly increased after the effluent was redirected to it (Figure B-9, Phase IV). Fluctuations occurred with rainfall events and valve adjustments that were used to regulate the volume of effluent each of the two trenches was receiving. An attempt was made to supply equal volumes of effluent to both trenches. When it appeared that one trench was receiving more effluent than the other, an adjustment was made, resulting in fluctuations.

For 30 days during the summer of 1984 (Days 637-674) all of the effluent was directed to the lower trench of the alternating system to decrease the ponding depth in the upper trench. The ponding depth in the upper trench gradually decreased, but it did not completely dry out (Figure B-9, Well 13). The ponding depth in the lower trench quickly increased (Figure B-9, Well 11) and remained high, even when both trenches were again put into service, throughout Phase IV.

The tensiometers installed in the existing system showed saturated soil conditions during the months when water was ponded in the system and unsaturated conditions when water was not ponded in the trenches (Figure B-10). The matric potentials during the summer of 1984 were not as great as those in 1983, indicating higher soil moisture contents. This may have been the result of greater precipitation and cooler temperatures during the summer of 1984.



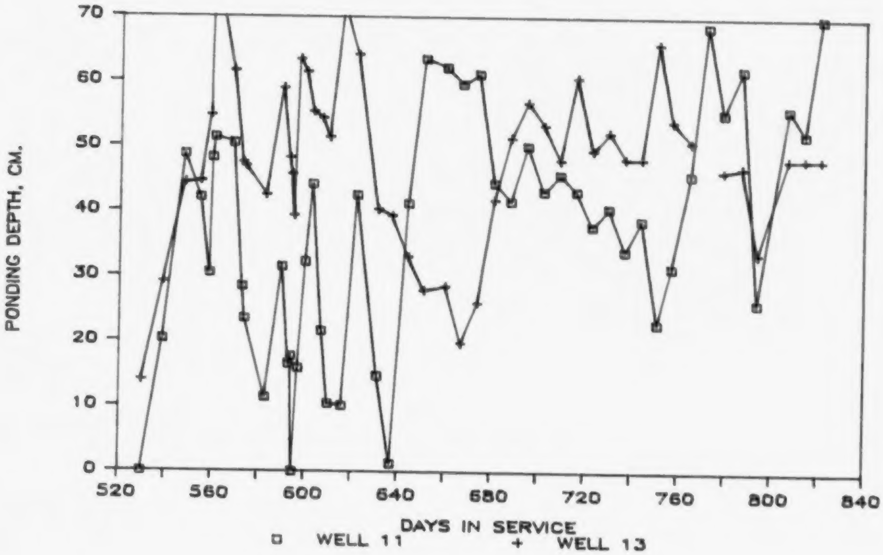


Figure B-9. Ponding depths in representative wells in the existing and alternating systems during Phase IV (2/21/84-12/21/84).

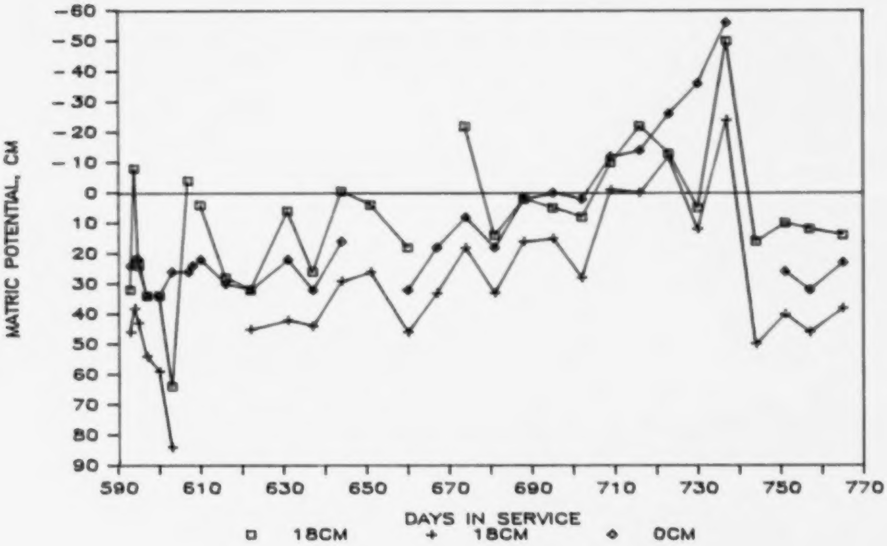


Figure B-10. The matric potential in the sidewalls of the existing system during part of Phase IV (4/17/84-11/23/84).

The soil matric potentials during the summer of 1984 (Phase IV) in the lower trench of the alternating system followed similar patterns to those during the summer of 1983 (Phase II). These potentials are shown in Figure B-11. The matric potentials in the upper trench of the alternating system during 1984 varied considerably from those in 1983. This is most likely the result of the higher effluent ponding depths during 1984. The soil below the trench was saturated throughout the Phase IV monitoring period (Figure B-12). The soil in the trench sidewall was also saturated through the 1984 summer monitoring period (Figure B-12).

Phase V

During the second winter of operation of the alternating system, surfacing of effluent occurred, indicating that additional trench capacity was required to dispose of the wastewater. Therefore, on December 21, 1984, effluent was directed to both the existing and the alternating systems at this site. Both systems were then required to treat and dispose of the effluent produced throughout the remainder of the monitoring period.

During Phase V, valves were used to regulate the volume of flow diverted to each system, and to each trench in the alternating system. The valves were adjusted weekly to correct for occasional localized overloading, especially during the winter months. Because of these volume changes, among other factors, the ponding depths in the trenches constantly fluctuated, as shown in Figure B-13.

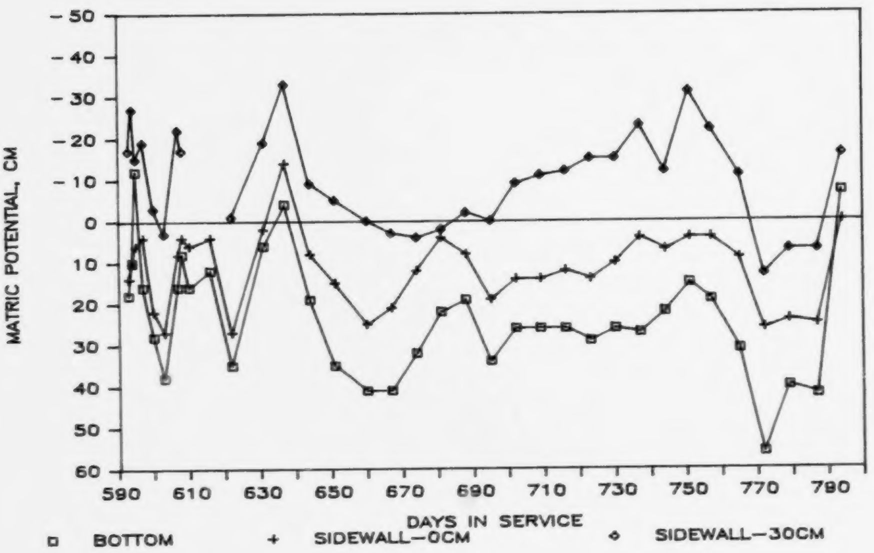


Figure B-11. The matric potential below the trench and in the trench sidewalls in the lower trench of the alternating system during part of Phase IV (4/17/84-11/23/84).

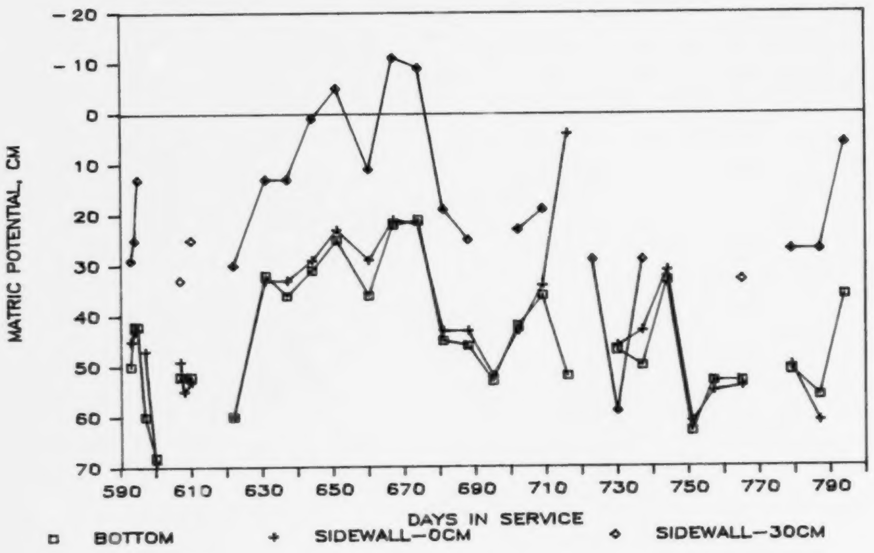


Figure B-12. The matric potential below the trench and in the trench sidewalls in the upper trench of the alternating system during Phase IV (4/17/84-11/23/84).

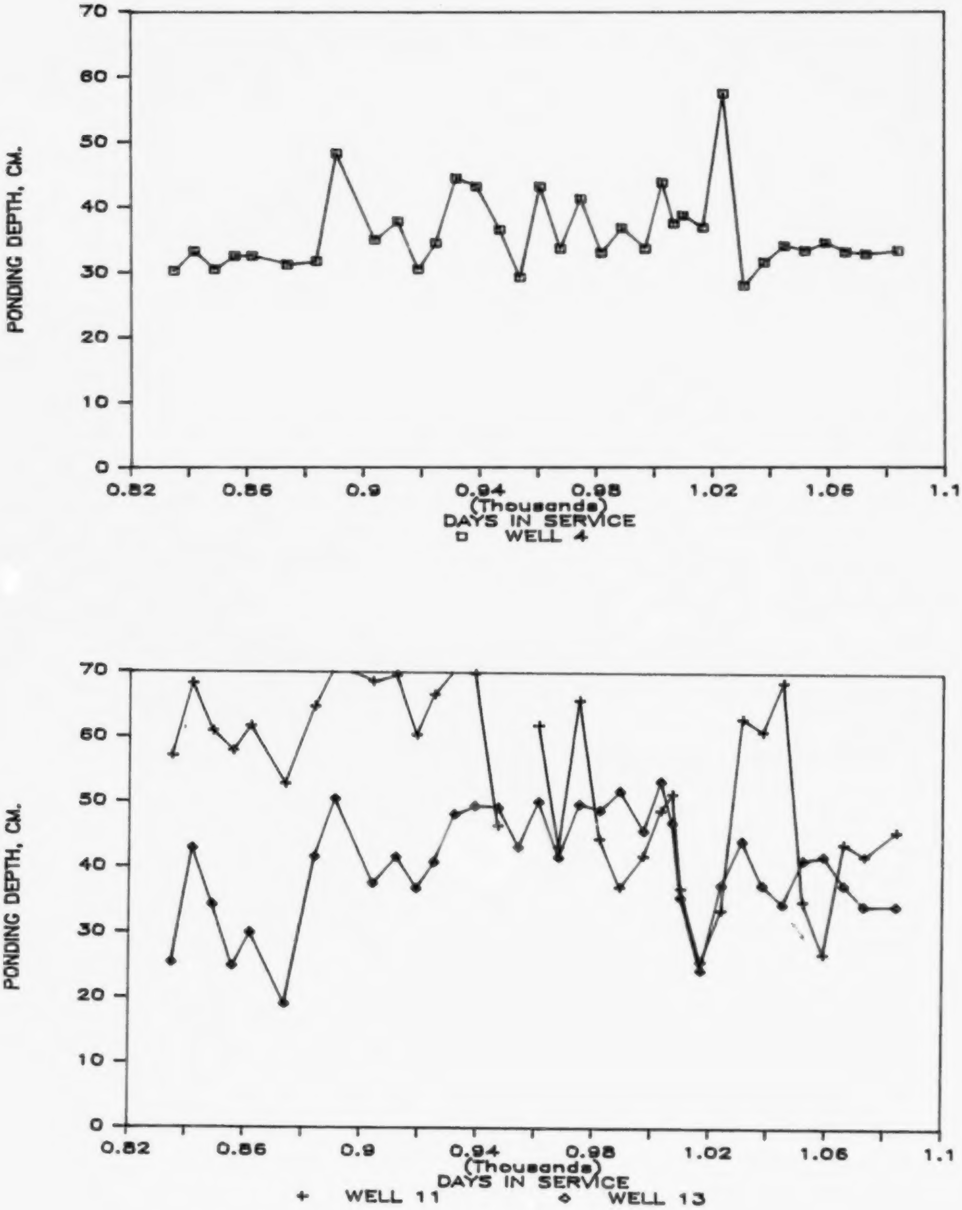


Figure B-13. Ponding depths in representative wells in the existing and alternating systems during Phase V (12/22/84-9/3/85).

During June 1985 (Days 1007-1024), several months before the end of the study period, the entire volume of effluent was directed to the existing system to determine if this system had rejuvenated sufficiently to dispose of all of the effluent during the summer months. Failure occurred in this system within 3 weeks, indicating that the system had not been sufficiently rejuvenated during the study. It was then necessary to keep both systems in service to prevent failure.

Precipitation Effects

The ponding depths in the trenches and the elevation of the groundwater table fluctuated with rainfall events throughout the study during the late fall, winter, and spring, when soil moisture contents were high and evapotranspiration rates were low. Figure B-14 shows the ponding depth and groundwater table depth below the soil surface from representative wells in the existing system, and the corresponding precipitation, during the spring of 1985 (Phase V).

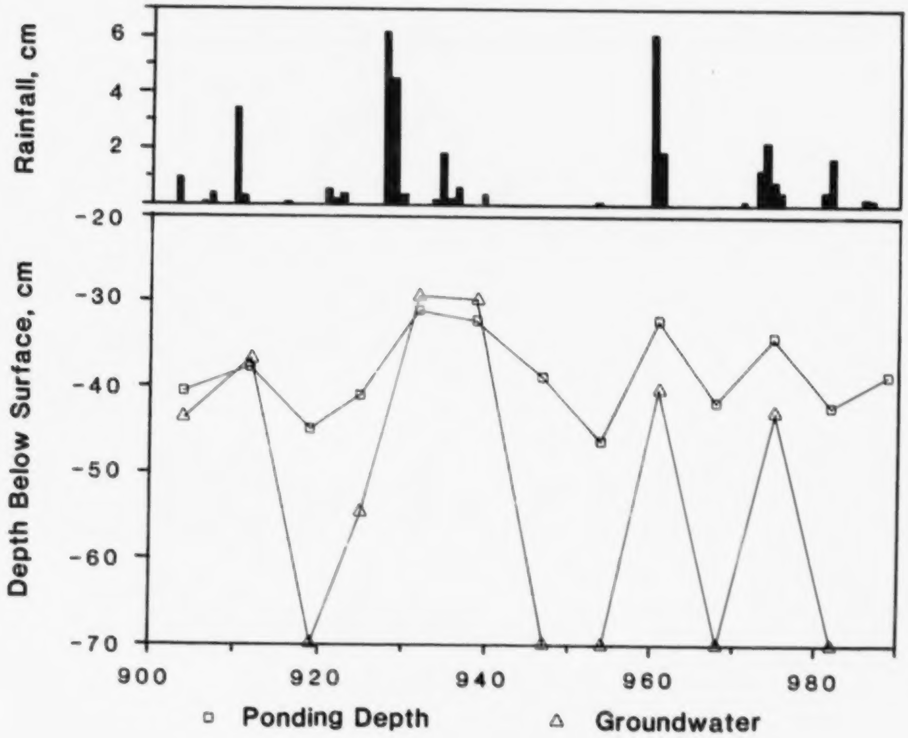


Figure B-14. Trench ponding depth and groundwater table depth below the soil surface in the existing system, and the corresponding precipitation levels, during the spring of 1985.

Effluent was directed to both systems during this time period, and the ponding depths in the existing system were at or near the full capacity of the trenches. During periods of high rainfall, both the ponding depth in the trenches and the groundwater table increased. The large groundwater table rise occurred rapidly after a rainfall event and then decreased rapidly. When the groundwater table was high, the trenches were surcharged, and the ponding depth in the trenches often exceeded the height of the trench.

This groundwater intrusion effect on the ponding depth in the existing system was also clearly demonstrated during the fall of 1983 (Phase II), when no effluent was applied to that system. During this period, the groundwater table rose above the trench bottom and inundated the trench with groundwater (Figure B-15). After the groundwater table fell, water remained in the trenches for several additional days. This slow drying out period was most likely caused by the clogging mat in the existing system.

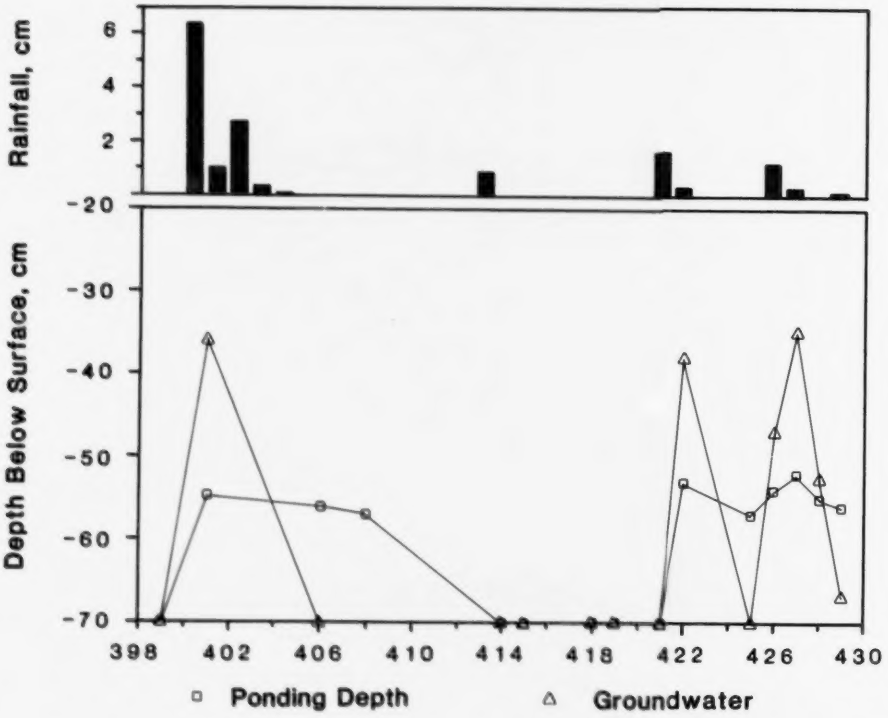


Figure B-15. Trench ponding depth and groundwater table depth below the soil surface in the existing system, and the corresponding precipitation levels, during fall, 1983.

Precipitation also resulted in minor fluctuations in the ponding depths in the alternating system trenches, but the fluctuations were not as great in magnitude as in the existing system, and the trenches were not often surcharged as the result of precipitation events. Groundwater did not appear to inundate the trenches in the alternating system, as was seen during the winter of 1984 when effluent was not applied to this system, and the trenches dried out (Phase III, Figure B-8).

The greater fluctuations in the existing system as the result of precipitation events and groundwater intrusion were most likely the result of the close proximity of the trenches to the bedrock. Approximately 1 m of soil is recommended between the infiltrative surface of the trench and bedrock (USEPA, 1980) because in areas with nonporous bedrock, such as in this study, groundwater can back up, or perch, above the bedrock. The trenches in the existing system did not have sufficient soil between them and the bedrock and were consequently inundated with the perched groundwater. In the alternating system, however, sufficient soil was present between the trenches and the bedrock, and inundation with groundwater was not observed.

Groundwater intrusion in the trenches of the existing system most likely prevented the total rejuvenation of the infiltrative surface. Biochemically reduced conditions prevail when groundwater or effluent is present in the trenches, and oxidation of the clogging mat cannot occur. During this study, the trenches in the existing system were dry for a maximum period of only 90 continuous days. This was probably not enough time to oxidize the clogging mat and adequately restore the system.

SUMMARY AND CONCLUSIONS

An alternating soil absorption field system was installed at a private residence in Hamilton County, Ohio, to correct a failure situation, and to try to restore a failed soil absorption field.

After intermittent periods of resting lasting no more than 90 days, insufficient rejuvenation occurred in the existing system to treat and dispose of the entire volume of wastewater produced at the site. Rejuvenation of the infiltrative surface was inhibited by the intrusion of groundwater into the existing trenches during the late fall, winter, and spring. Groundwater that was perched above the bedrock inundated the trenches because of the close proximity of the trenches to the bedrock. This study clearly shows the importance of maintaining a suitable isolation distance between the trench bottom and an impervious horizon, such as bedrock.

Although the existing system was not rejuvenated sufficiently during this study to absorb the entire volume of wastewater produced, the additional 32 m of trench installed in the alternating system did improve the failure situation, and continuous surfacing of effluent did not occur during the time period of this study.

From this study it appears that an alternating soil absorption field system is not a suitable alternative system in soil with fluctuating high groundwater tables.

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Appendix C

PERFORMANCE OF A LOW-PRESSURE PIPE SOIL ABSORPTION FIELD INSTALLED IN AN ILLINOIAN TILL SOIL*

David Effert Margaret Cashell

INTRODUCTION

Soils with a seasonally or permanently high water table, low permeability, or shallow depth to bedrock or other impermeable layer are not considered suitable for the installation of a conventional septic tank and soil absorption field. Alternatives to the conventional onsite disposal system have been proposed for problem areas, and many designs have been evaluated, but no single alternative works well in all climates and where there are multiple site limitations.

The low-pressure pipe (LPP) soil absorption field has been found to be an acceptable alternative onsite wastewater disposal system in North Carolina (Carlile, 1979; Floyd, 1982; Hargett, 1984). Questions still remain, however, concerning the system's adaptability in colder climates and its performance when installed in locations that do not meet site and soil suitability requirements established by the Triangle J. Council of Governments (1979).

This research evaluates the performance of a LPP soil absorption field installed in a temperate climate in an area with multiple site limitations. The three objectives of the research were:

1. To determine if an LPP soil absorption field would contaminate groundwater when installed in a soil having a seasonally high water table.
2. To determine if the LPP soil absorption field is an effective all-season alternative when installed in a temperate climate.

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* Abstract in: Proc. ASAE Speciality Conference on Environmental Engr. Paper No. 85-2632. American Society of Agricultural Engineers. Chicago, Illinois. 1984.

3. To estimate the long-term loading rate of an LPP system installed in a Rossmoyne soil.

MATERIALS AND METHODS

The performance of the LPP soil absorption field was studied at the Cincinnati Center's field research station located in Clermont County, Ohio. The system was designed and installed based on the construction procedures recommended by the Triangle J. Council of Governments (1979).

Site and Soils

The LPP soil absorption field was installed on a 10%, east-facing side slope in a moderately eroded Rossmoyne silt loam soil (Aquic Fragiudalf, fine-silty, mixed, mesic). This soil formed in 46 to 100 cm (18 to 39 in) of silt mantle and in the underlying weathered glacial till of Illinoian age (Lerch et al., 1975).

A representative profile of the soil is presented in Table C-1. The depth to mottles ranges from 36 to 64 cm (14 to 25 in) in a Rossmoyne soil. Permeability is slow, and the fragipan can restrict the downward movement of water. Iron-manganese concretions and stains are common in and below the fragipan (Lerch et al., 1975).

TABLE C-1. SOIL CHARACTERISTICS AT THE LOCATION OF THE LPP SYSTEM

Depth	Color	Description
23-46 cm	yellowish brown (10YR5/4)	silty clay loam with subangular blocky structure
46-100 cm	yellowish brown (10YR5/4)	clay loam with brittle fragipan
100-218 cm	yellowish brown (10YR5/4) light brownish grey (10YR6/2)	mottled clay loam
218-330 cm	yellowish brown (10YR5/4) olive brown (2.5Y4/4)	massive silt loam and loam

The soil in this study differed from the representative description of a Rossmoyne soil in that the fragipan was only very weakly expressed and did not appear to greatly impede the downward movement of water. Also, mottles occurred at approximately 38 cm (15 in) in the upslope position and 15 cm (6 in) in the downslope position of the system, and bedrock occurred at a depth ranging from 1.5 to 3 m (5 to 10 ft) over the area of the system.

The Triangle J. Council of Governments (1979) recommends a minimum soil depth of 76 cm (30 in) from the finished surface to the seasonally high water table. The occurrence of 2-chroma mottles at 15 to 38 cm (6 to 15 in) suggests that the soils in this study did not meet this criteria. The 10% slope of the site is the maximum slope that is recommended by the Triangle J. Council of Governments (1979) for the installation of an LPP system.

Installation Procedure

The low-pressure pipe soil absorption field consisted of five trenches that were 12.2 m (40 ft) long and spaced 1.5 m (5 ft) on center for a total of 61 linear meters (200 ft) of trench (Figure C-1). The trenches were 20 cm (8 in) wide and approximately 28 cm (11 in) deep and were installed on the contour using a trenching machine as recommended by the Triangle J. Council of Governments (1979). Eighteen centimeters (7 in) of gravel was placed in the bottom of the trench; a 31.8-mm (1.25-in) diameter PVC distribution pipe was placed on top of the gravel; and the pipe was covered with a thin layer of gravel. Typar®, a brand of polyester fabric, was laid above the gravel, and the entire trench was covered with 7.6 cm (3 in) of soil to bring the trench up to grade. An additional 10.2 cm (4 in) of soil was mounded above the trench to prevent surface water from ponding and to allow for future settling.

Pressure distribution of septic tank effluent was employed using design criteria presented by Otis (1981). In order to obtain even distribution within the five trenches on the 10% slope, the distribution pipe required various orifice diameters and separation distances. The pressure distribution system used a 3785-l (1000-gal) pumping chamber equipped with a properly sized submersible pump activated by an electronic timer. The timer activated the pump once a day and provided a single dose of septic tank effluent to the LPP trenches.

Monitoring Procedures

Observation wells were installed in each of the five trenches (Figure C-2). The wells consisted of 5.1-cm (2-in) diameter PVC pipe perforated in the lower section to maintain hydraulic continuity with the water in the trench. The ponding depth was measured once a week prior to a dosing event.

Five additional pairs of PVC wells were installed near the LPP soil absorption field to collect groundwater data. A pair of control wells (S1 and D1), located 1.5 m (5 ft) upslope from the LPP trenches, were installed to obtain background groundwater data. The other four pairs of wells were located in the middle of the system (S2 and D2) and 1.5 m (5 ft) downslope (S3, D3, S4, D4, S5, and D5) from the LPP trenches (Figure C-1). The wells were installed in pairs to obtain groundwater data from two different depths at the same location; one well was installed to a depth of 61 cm (24 in) (S group), while the second well was installed to a depth of 140 to 152 cm (55 to 60 in) (D group).

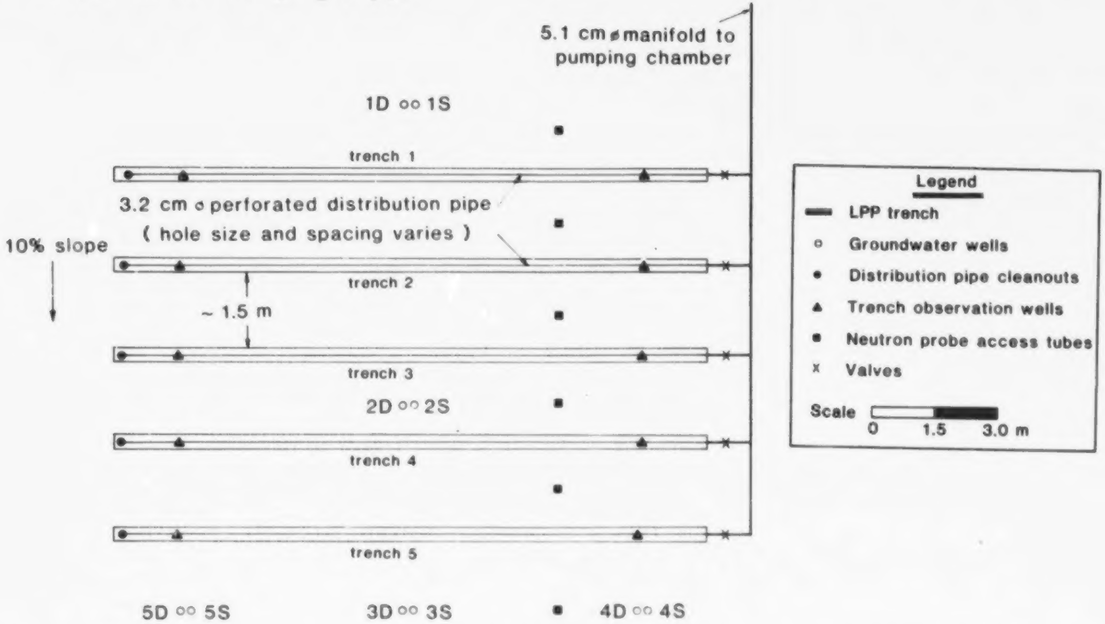
5.1 cm ø manifold to pumping chamber
1D oo 1S
trench 1
3.2 cm ø perforated distribution pipe
(hole size and spacing varies)
10% slope
trench 2
~ 1.5 m
trench 3
2D oo 2S
trench 4
trench 5
5D oo 5S
3D oo 3S
4D oo 4S
Legend
LPP trench
Groundwater wells
Distribution pipe cleanouts
Trench observation wells
Neutron probe access tubes
Valves
Scale
0
1.5
3.0 m

Figure C-1. Diagram of the low-pressure pipe soil absorption field and monitoring equipment locations.

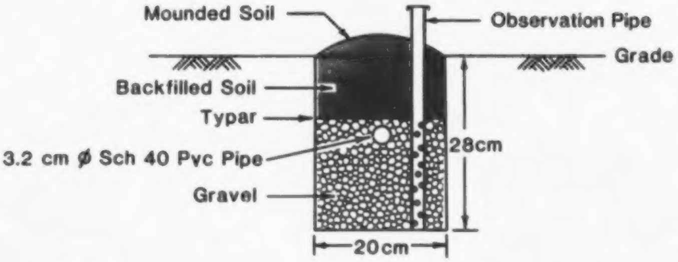
Mounded Soil
Observation Pipe
Grade
Backfilled Soil
Typar
3.2 cm Ø Sch 40 Pvc Pipe
Gravel
28cm
20cm

Figure C-2. Cross section of a typical low-pressure pipe soil absorption trench showing the placement of an observation well.

A groundwater monitoring program was initiated in October 1983 and continued for one year. The program included measuring the distance from the ground surface to the water table prior to a dosing event and collecting groundwater samples. The sampling procedure included pumping the observation wells dry prior to a dosing event using a small portable peristaltic pump and sterile Tygon® tubing. Following the dosing event, the wells were allowed to recharge for a period of 3 to 4 hours before three groundwater samples from each well were collected. Fecal coliform samples were collected in sterile glass bottles, and the samples analyzed for chemicals were collected in nonsterile containers. Samples of the septic tank effluent applied to the LPP soil absorption field were also collected during this sampling program. All of the samples were refrigerated within 1 hour after collection, and the samples used for chemical analyses, excluding chlorides, were preserved with 1 ml of H_2SO_4 per liter of sample.

System Operation

Septic tank effluent was initially applied to the LPP trenches in a single daily dose of 303 l (80 gal), which was equivalent to an LPP loading rate of 4.1 lpd/sq m (0.1 gpd/sq ft). The Triangle J. Council of Governments (1979) calculates the loading rate based on the total surface area the LPP soil absorption field covers; in this case the area is 12.1 X 6.1 m (40 X 20 ft). The loading rate was later adjusted up or down (Table C-2) to reflect the seasonal changes in the system's ability to absorb effluent as determined by the depth of septic tank effluent ponding in the trenches.

TABLE C-2. LOW-PRESSURE PIPE SOIL ABSORPTION FIELD LOADING RATE HISTORY

Time period	Peak flow rate, lpd	Peak loading rate, lpd/sq m
Sep 1983 - Apr 1984*	303 (80)†	4.1§ (0.10)**
Apr - May 1984	454 (120)	6.1 (0.15)
May - Jul 1984††	568 (150)	7.7 (0.19)
Jul - Sep 1984††	757 (200)	10.2 (0.25)
Sep - Oct 1984	568 (150)	7.7 (0.19)
Oct 1984 - Aug 1985	303 (80)	4.1 (0.10)

* The system occasionally failed during this time period when the seasonal water table was near the ground surface.

† gpd.

§ Based on an absorption area of 74 sq m.

** gpd/sq ft, based on an absorption area of 800 sq ft.

†† Electrical malfunctions occurred during this period, and the LPP system was occasionally taken out of service.

RESULTS AND DISCUSSION

Septic Tank Effluent Concentration

Results of the analysis of the septic tank effluent applied to the LPP trenches (Table C-3) show that, in general, its strength was slightly weaker than concentrations reported in the literature (Otis and Boyle, 1976).

TABLE C-3. CONCENTRATION OF THE SEPTIC TANK EFFLUENT APPLIED TO THE LOW-PRESSURE PIPE SOIL ABSORPTION FIELD

Parameter	Number of samples	Mean	Median	Range
		mg/l		
NH_4-N	58	16.5	17.2	2.9-30.0
NO_2+NO_3-N	58*	0.1	0.1	<0.1-0.9
TKN	58	20.1	20.6	3.1-44.1
Total-P	58	7.6	8.4	0.9-17.1
COD	58	86.0	86.0	21-218
Chloride	74	75.9	61.2	5.0-497.0

* Thirty samples had a concentration less than the detectable limit of 0.1 mg/l. The mean and median was calculated by assuming a value of 0.1 mg/l for these samples.

Groundwater Monitoring

A number of wastewater constituents may be found in groundwater as a result of contamination from a nearby soil absorption field (Cogger and Carlile, 1984; Reneau, 1977, 1979; Uebler, 1984). Contaminants typically monitored include various forms of nitrogen, phosphorus, fecal coliforms, and chloride. Of these, only chloride undergoes no biological transformations, and it is not adsorbed onto soil particles. As a result, chloride is a useful tracer or indicator of septic tank effluent movement through soil (Canter and Knox, 1984).

The chloride concentration of the groundwater steadily increased over time (Figure C-3). On any given day, the chloride concentration of the control well (D1) was consistently and noticeably lower than well D3 located downslope from the trenches (Figure C-3). Average values show the same trend. The control wells had an average chloride concentration of 7.2 mg/l, while the wells located downslope from the trenches had average concentrations ranging from 21.3 to 44.3 mg/l (Table C-4). The movement of septic tank effluent away from the trenches appeared to be in a downslope direction, as shown by the higher chloride concentration in the downslope wells. With respect to chloride, the groundwater obtained from the wells appeared to be influenced by the application of the septic tank effluent into the LPP trenches.

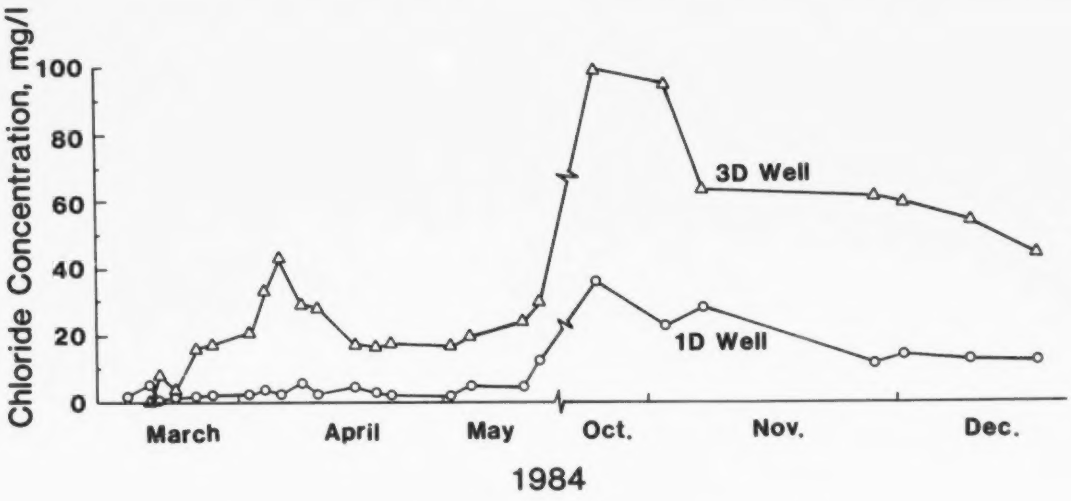

Figure C-3. Chloride concentration of groundwater obtained from the control (1D) and downslope (3D) monitoring wells during the spring and fall of 1984.

TABLE C-4. CHLORIDE CONCENTRATION OF GROUNDWATER OBTAINED FROM OBSERVATION WELLS LOCATED NEAR THE LOW-PRESSURE PIPE SOIL ABSORPTION FIELD

Location	Notes	Number of samples	Mean	Median	Range
Well 1S	Control well	25 (0)*	7.3 (A)†	2.7	0.9-27.3
Well 1D	Control well	26 (1)	7.2 (A)	3.2	<0.1-36.9
Well 3S	Downslope 1.5 m	26 (1)	36.4 (B)	23.9	8.4-140.0
Well 3D	Downslope 1.5 m	32 (2)	35.9 (B)	29.3	<0.1-99.5
Well 4S	Downslope 1.5 m	18 (0)	31.4 (B)	18.6	3.5-93.9
Well 4D	Downslope 1.5 m	24 (0)	21.3 (A)	14.7	1.6-103.0
Well 5S	Downslope 1.5 m	15 (0)	44.3 (B)	12.5	2.4-136.0
Well 5D	Downslope 1.5 m	22 (0)	34.2 (B)	12.1	4.0-124.0

* Number of samples having a concentration less than the detectable limit of 0.1 mg/l. Means and medians were calculated by assuming a value of 0.1 mg/l for these samples.

† Means with the same letter are not significantly different at the 95% confidence level as determined by Duncan's Multiple Range Test.

The nitrogen concentration of the groundwater obtained from observation wells located upslope and downslope from the LPP system are given in Table C-5. Virtually all of the samples collected had extremely low ammonium, nitrate + nitrite, and total Kjeldahl nitrogen concentrations when compared with other values reported in the literature for groundwater collected near soil absorption fields (Cogger and Carlile, 1984; Reneau, 1977, 1979; Uebler, 1984).

TABLE C-5. AMMONIUM-N, NITRATE + NITRITE-N, AND TOTAL KJELDAHL-N CONCENTRATIONS IN GROUNDWATER FROM OBSERVATION WELLS LOCATED NEAR THE LOW-PRESSURE PIPE SOIL ABSORPTION FIELD

Parameter	Location	Number of samples	Mean	Median	Range
NH_4-N, mg/l	Well 1S	26 (18)*	0.2 (A)†	0.1	<0.1-0.8
	Well 1D	31 (14)	0.4 (A)	0.1	<0.3-3.0
	Well 3S	26 (19)	0.2 (A)	0.1	<0.1-0.8
	Well 3D	26 (17)	0.1 (A)	0.1	<0.1-0.6
	Well 4S	6 (5)	0.1 (A)	0.1	<0.1-0.1
	Well 4D	9 (8)	0.1 (A)	0.1	<0.1-0.2
	Well 5S	3 (2)	0.1 (A)	0.1	<0.1-0.1
	Well 5D	9 (7)	0.1 (A)	0.1	<0.1-0.3
NO_2+NO_3-N, mg/l	Well 1S	26 (16)	0.4 (A)	0.1	<0.1-6.1
	Well 1D	31 (13)	0.6 (A)	0.1	<0.1-6.1
	Well 3S	26 (20)	0.1 (A)	0.1	<0.1-0.3
	Well 3D	26 (16)	0.1 (A)	0.1	<0.1-0.3
	Well 4S	6 (5)	0.1 (A)	0.1	<0.1-0.2
	Well 4D	9 (6)	0.1 (A)	0.1	<0.1-0.2
	Well 5S	3 (1)	0.3 (A)	0.3	<0.1-0.6
	Well 5D	9 (6)	0.8 (A)	0.1	<0.1-6.1
TKN, mg/l	Well 1S	26 (4)	1.0 (A)	0.9	<0.1-2.7
	Well 1D	31 (5)	1.1 (A)	0.9	<0.1-8.2
	Well 3S	26 (5)	1.0 (A)	1.1	<0.1-2.8
	Well 3D	26 (6)	0.7 (A)	0.4	<0.1-2.3
	Well 4S	6 (3)	0.4 (A)	0.4	<0.1-0.8
	Well 4D	9 (3)	0.3 (A)	0.2	<0.1-0.6
	Well 5S	3 (1)	0.4 (A)	0.5	<0.1-0.7
	Well 5D	9 (0)	0.7 (A)	0.2	<0.2-2.7

* Number of samples having a concentration less than the detectable limit of 0.1 mg/l. Means and medians were calculated by assuming a value of 0.1 mg/l for these samples.

† Means with the same letter for the same parameter are not significantly different at the 95% confidence level as determined by Duncan's Multiple Range Test.

The median ammonium concentration was 0.1 mg/l for all of the wells, with means ranging from <0.1 to 0.4 mg/l. This small range indicates that throughout the sampling period there was little change in the ammonium concentration of the groundwater in the sampling wells.

Nitrate + nitrite concentrations followed a similar pattern throughout the study with mean and median values low and the range of values small. Median values ranged from 0.1 to 0.3 mg/l while means ranged from <0.1 to 0.8 mg/l.

The total Kjeldahl nitrogen concentrations of the groundwater samples were also low. Means ranged from 1.0 to 1.2 mg/l in the control wells and from 0.3 to 1.0 mg/l in the wells located downslope from the trenches (Table C-5).

There was no statistical difference between the ammonium-N, nitrate + nitrite-N, and total Kjeldahl nitrogen concentrations in the groundwater obtained from the control wells vs. the groundwater obtained from the wells located downslope from the trenches (Table C-5). The groundwater obtained from the wells, at least with respect to ammonium, nitrate + nitrite, and total Kjeldahl nitrogen, was probably not influenced by the presence of the LPP soil absorption field during this study.

Average total phosphorus concentrations are summarized in Table C-6. Mean and median concentrations exceeded 1.0 mg/l in wells 3S and 3D, but the data were random and did not show any trend over time or from season to season. Total phosphorus concentrations from the control wells (1S and 1D) were relatively high, with means of 0.9 and 0.8 mg/l, respectively. Prior to its use as a field research station, the property where the LPP system was installed was used for agricultural purposes. The phosphorus concentration of the groundwater in both the control and downslope wells greatly exceed 0.01 to 0.1 mg/l which is the typical phosphorus concentration in agricultural soil solutions (Brown et al., 1977).

TABLE C-6. TOTAL PHOSPHORUS CONCENTRATION AND FECAL COLIFORM NUMBERS IN GROUNDWATER FROM OBSERVATION WELLS LOCATED NEAR THE LOW-PRESSURE PIPE SOIL ABSORPTION FIELD.

Parameter	Location	Number of samples	Mean	Median	Range
Total-P, mg/l	Well 1S	26 (7)*	0.9 (A)†	0.9	<0.1-3.2
	Well 1D	31 (7)	0.8 (A)	0.5	<0.1-6.5
	Well 3S	26 (5)	1.3 (A)	1.1	<0.1-5.9
	Well 3D	25 (5)	1.2 (A)	1.1	<0.1-4.3
	Well 4S	6 (4)	0.2 (A)	0.1	<0.1-0.4
	Well 4D	9 (2)	0.4 (A)	0.3	<0.1-0.9
	Well 5S	3 (2)	0.1 (A)	0.1	<0.1-0.2
	Well 5D	9 (4)	0.4 (A)	0.3	<0.1-1.1
Fecal coliforms, #/100 ml	Well 1S	8 (4)§	15 (A)	2	<2-79
	Well 1D	8 (5)	<2 (A)	<2	<2-8
	Well 3S	9 (6)	20 (A)	<2	<2-130
	Well 3D	8 (6)	<2 (A)	<2	<2-8
	Well 4S	3 (2)	6 (A)	<2	<2-17
	Well 4D	3 (2)	6 (A)	<2	<2-17

* Number of samples having a concentration less than the detectable limit of 0.1 mg/l. Means and medians were calculated by assuming a value of 0.1 mg/l for these samples.

† Means with the same letter for the same parameter are not significantly different at the 95% confidence level as determined by Duncan's Multiple Range Test.

§ Number of samples having less than 2 fecal coliforms/100 ml (MPN method). Means and medians were calculated by assuming a value of 0/100 ml.

With respect to the total phosphorus concentration, there was no statistical difference between the groundwater collected from the control wells and the groundwater collected from the downslope wells. The LPP soil absorption field does not appear to be contributing to the total phosphorus concentrations in the downslope wells. Canter and Knox (1984), Carlile et al. (1981) and Brown et al. (1977) also reported that the presence of an operating soil absorption field had little impact on the phosphorus concentration of groundwater obtained from observation wells located near the field.

Results of a limited number of fecal coliform tests (Most Probable Number) run on the groundwater samples are reported in Table C-6. Mean values ranged from <2 to 20 coliforms per 100 ml; individual sample counts ranged from <2 to 130 per 100 ml. Sixty-three percent of the samples from all of the wells had less than the detectable limit of 2 fecal coliforms per 100 ml.

Pressure Distribution and Effluent Ponding in the Trenches

Effluent ponding in the LPP trenches prior to a dosing event was common, and the goal of resting the trenches as a result of using pressure distribution and a single daily dosing event was not always achieved. Ponding was seasonal, and during the late fall, winter, and spring it occurred to a depth of 10 cm (4 in) or more in all the trenches. Trench #5, located downslope, consistently had the greatest depth of ponding, while trench #1, located upslope, consistently had the least.

Hargett (1984) also observed that downslope trenches in LPP systems stayed ponded more often than upslope trenches. He suggested two possible explanations. First, the downslope trenches may be subjected to wetter background conditions, owing to lateral flow from the upslope trenches. This may result in less absorption by the downslope trenches. A second possibility is that the difference in ponding depths may be the result of nonuniform distribution. In this study, it was determined that uniform distribution was achieved and that the depth of effluent ponding was due to the different absorption rates of each trench. The downslope trenches may have been influenced by the trenches upslope from them, resulting in their having a lower absorption rate as suggested by Hargett (1984). Nonuniform soil properties within the area of the soil absorption field may also have caused unequal absorption.

Regardless of the cause, nonuniform absorption did occur, and this resulted in localized overloading of the trenches and effluent ponding. It is therefore recommended that a valve be placed between the manifold and the distribution pipe of each trench so that the loading rate can be adjusted. Proper adjustment of the valves could result in the individualized loading of the trenches based on their ability to absorb effluent during those times when ponding occurs. The valves could also be used to take a particular trench out of service for long-term resting.

The relationship between the distance to groundwater measured in observation wells located near the LPP system and the depth of ponding in the trenches is shown in Figure C-4. The control Well (1D), located 1.5 m (5 ft) upslope from the trenches, consistently had a greater distance to groundwater from grade than well 2D which was located between trenches 3 and 4. This suggests that the LPP system caused the groundwater to mound beneath the trenches such that it was closer to grade than it would have been had the trenches not been there.

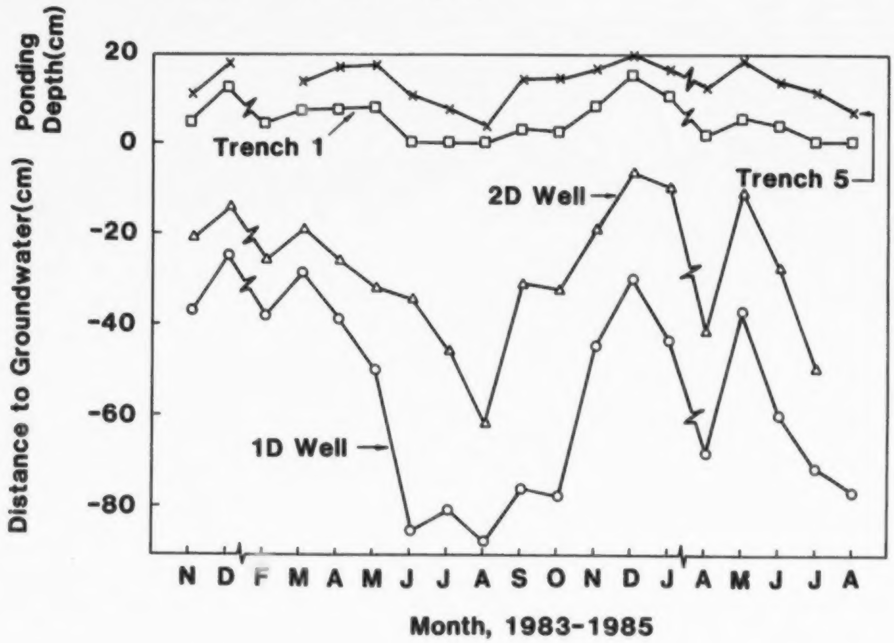

Figure C-4. Relationship between effluent ponding in the trenches and the distance from grade to the water table.

Other research at the Cincinnati Center's Field Research Station (Cashell and Effert, 1985) has determined that in an area not influenced by the LPP system, soil moisture below approximately 1 m (3.3 ft), as measured with a neutron probe, was very close to saturation throughout the year. Because the soil was nearly saturated, only a small amount of additional moisture was required to develop a water table, and this moisture may have been provided by the effluent from the LPP trenches. The groundwater measurements and neutron probe data suggest that the LPP system contributed to the formation of the water table. Therefore, maintaining a 76 cm (30 in) separation distance between the groundwater and the bottom of the trench, as recommended by the Triangle J. Council of Governments (1979), may not be possible throughout most of the year in a Rossmoyne soil.

The single dose of wastewater that was applied to the trenches daily was not always absorbed by the soil before another dosing event. Therefore, resting of the trenches between dosing events, a goal of using pressure distribution, was not achieved in this study.

Long-Term Loading Rate

Effluent absorption by the low-pressure pipe soil absorption field was variable and dependent on the season, precipitation, and evapotranspiration rates. Loading rates in this study ranged from 4.1 to 10.2 lpd/sq m (0.1 to 0.25 gpd/sq ft), while recommended design loading rates range from 2.0 to 20.1 lpd/sq m (0.05 to 0.50 gpd/sq ft) (Triangle J. Council of Governments, 1979). A long-term loading rate of 4.1 lpd/sq m (0.1 gpd/sq ft) for the soil and site conditions present in this study appears reasonable during most of the year. However, seasonal failure may occur when the water table is near the surface of the ground.

Freezing Potential

The low-pressure pipe soil absorption field was originally developed and used in North Carolina where the winters are mild and the soil rarely freezes to any significant depth for extended periods of time. The frost line is deeper in northern states, however, and freezing of the LPP system may be a potential problem because of the shallow trenches.

In January 1984, the LPP system was taken out of service to repair the leaking pumping chamber. At that time, one of the trenches froze and stayed frozen for 4 weeks. Temperatures moderated by February, and the trench thawed out before septic tank effluent was again applied, and it was not possible to determine whether the application of effluent would have thawed the frozen trench. During the winter of 1984-1985, the system was in continuous service, and the trenches did not freeze. If the LPP soil absorption field is not used continuously during winter, the shallow trenches may be subject to freezing. More research on this topic is suggested before specific recommendations can be made concerning the freezing potential of this design.

SUMMARY AND CONCLUSIONS

A low-pressure pipe soil absorption field was installed in an Illinoian till soil in a temperate climate and monitored for a 2-year period.

Results of a groundwater monitoring program near the LPP soil absorption field showed that chloride was moving downslope from the trenches, but statistically there was no difference in the concentrations of ammonium-N, nitrite + nitrate-N, total Kjeldahl nitrogen, total phosphorus, and fecal coliforms between the groundwater in the downslope wells vs. the control wells. The application of septic tank effluent into the soil through the LPP trenches had little or no effect on the groundwater quality in the observation wells located downslope from the system during this study.

The LPP trenches received septic tank effluent at loading rates ranging from 4.1 to 10.2 lpd/sq m (0.1 to 0.25 gpd/sq ft). During the dry summer months when evapotranspiration and absorption were at their peak, a period of resting between dosing events occurred in some of the trenches even at the highest loading rate. During the wetter winter and spring months when the loading rates were at their lowest, effluent was continuously ponded in the trenches. The use of pressure distribution and dosing did not achieve the goal of resting the trenches between dosing events.

When installed in temperate climates, the shallow trenches of the LPP soil absorption field may freeze. Additional research is necessary to determine the probability of freezing in various geographic regions of the country.

The low-pressure pipe soil absorption field monitored in this study was installed in an area that historically has been shown to be unsuitable for conventional onsite wastewater disposal. Other research at this location has confirmed the fact that for as much as 60% of the year, conventional trenches do not function properly due to the presence of a seasonally high water table (Effert and Cashell, 1986). The construction of any onsite wastewater treatment system is not recommended at sites having the limitations present at this location. If onsite treatment is used, an LPP soil absorption field is recommended and a 4.1 lpd/sq m (0.1 gpd/sq ft) long-term loading rate or lower is suggested for a Rossmoyne soil.

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Appendix D

A COMPARATIVE STUDY OF THREE SOIL ABSORPTION TRENCH DESIGNS INSTALLED IN AN ILLINOIAN TILL SOIL*

David Effert Margaret Cashell

INTRODUCTION

Onsite wastewater treatment systems are frequently constructed in areas where site or soil conditions are unsuitable. Too often pressures to develop the land are great and site limitations are ignored, and the installation of onsite wastewater treatment systems takes place without regard for local conditions. This study reports on the performance of three onsite soil absorption trench designs that were intentionally installed in an area which historically has been shown to be unsuitable for onsite wastewater treatment. The study was undertaken to compare the performance of the three soil absorption trench designs in order to achieve the following objectives:

1. To determine if the Avonburg Soil Series may be suitable for the absorption, treatment, and disposal of septic tank effluent.
2. To determine which of the three soil absorption trench designs, when installed in an Avonburg soil, absorbs the greatest volume of septic tank effluent during various seasons of the year.
3. To compare the performance of SB2® gravelless distribution tubing to the performance of a conventional gravel filled trench.
4. To determine if perimeter drains are an effective solution to a seasonally high water table, which is typically associated with the Avonburg soil series.

MATERIALS AND METHODS

Description of the Soils

The trenches were installed on a nearly level upland landscape position in an Avonburg silt loam soil (Aeric Fragiaqualf). This soil formed in Wisconsinan loess and in the underlying weathered till of

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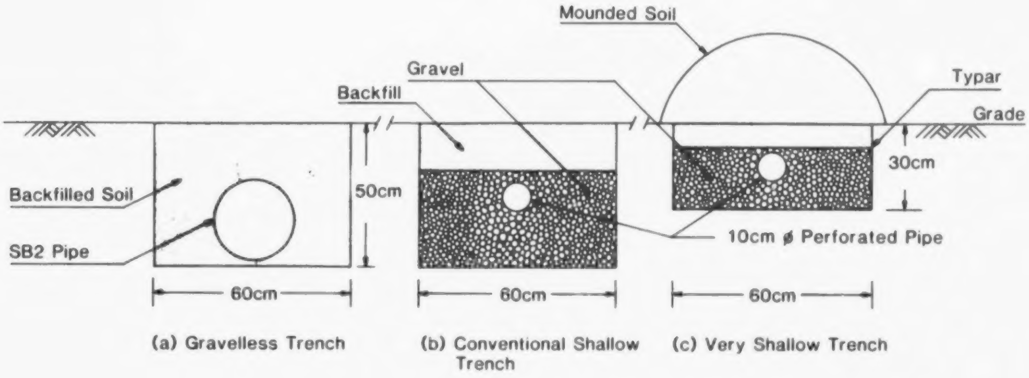
Illinoian age. A clay loam fragipan was present from approximately 68 to 170 cm (27 to 67 in), but it was not well expressed in some areas. Interbedded Ordovician limestone and shale bedrock occurred within 2.5 m (8.2 ft) over the site.

Permeability is very slow in Avonburg soils, and a high water table is present during the late fall, winter, and spring (Lerch et al., 1975). These conditions can limit the use of this soil for onsite wastewater disposal.

Trench Designs

The three trench designs were installed and evaluated at the Cincinnati Center's Field Research Station located in southwestern Ohio. The trenches, each 6.1 m (20 ft) long, were constructed next to each other and separated by a distance of 6.1 m (20 ft) as shown in Figure D-1.

Cross sections of the three trenches are presented in Figure D-2. The conventional trenches were 50 cm (20 in) deep and 60 cm (24 in) wide and filled with 20 cm (8 in) of standard 2.5- to 5.0-cm (1- to 2-in) diameter washed lateral gravel. Above the gravel, 10-cm (4-in) diameter perforated distribution pipe was laid with an additional 5 cm (2 in) of gravel placed over the pipe, resulting in a total gravel depth of 36 cm (14 in). The gravel was covered with Typar®, a brand of polyester fabric, before the trench was backfilled with 15 cm (6 in) of soil.



(a) Gravelless Trench (b) Conventional Shallow Trench (c) Very Shallow Trench

Figure D-2. Cross section of the three trench designs.

The shallow trenches were installed by placing 7.6 cm (3 in) of standard lateral gravel into the bottom of 30 cm (12 in) deep and 60 cm (24 in) wide trenches. Perforated distribution pipe, 10 cm (4 in) in diameter, was laid above the gravel and it was covered with additional gravel to a depth of 2.5 cm (1 in) above it for a total gravel depth of 20 cm (8 in). A layer of Typar® was placed above the gravel before it was backfilled with 2.5 cm (4 in) of soil to bring the trench up to grade. An additional

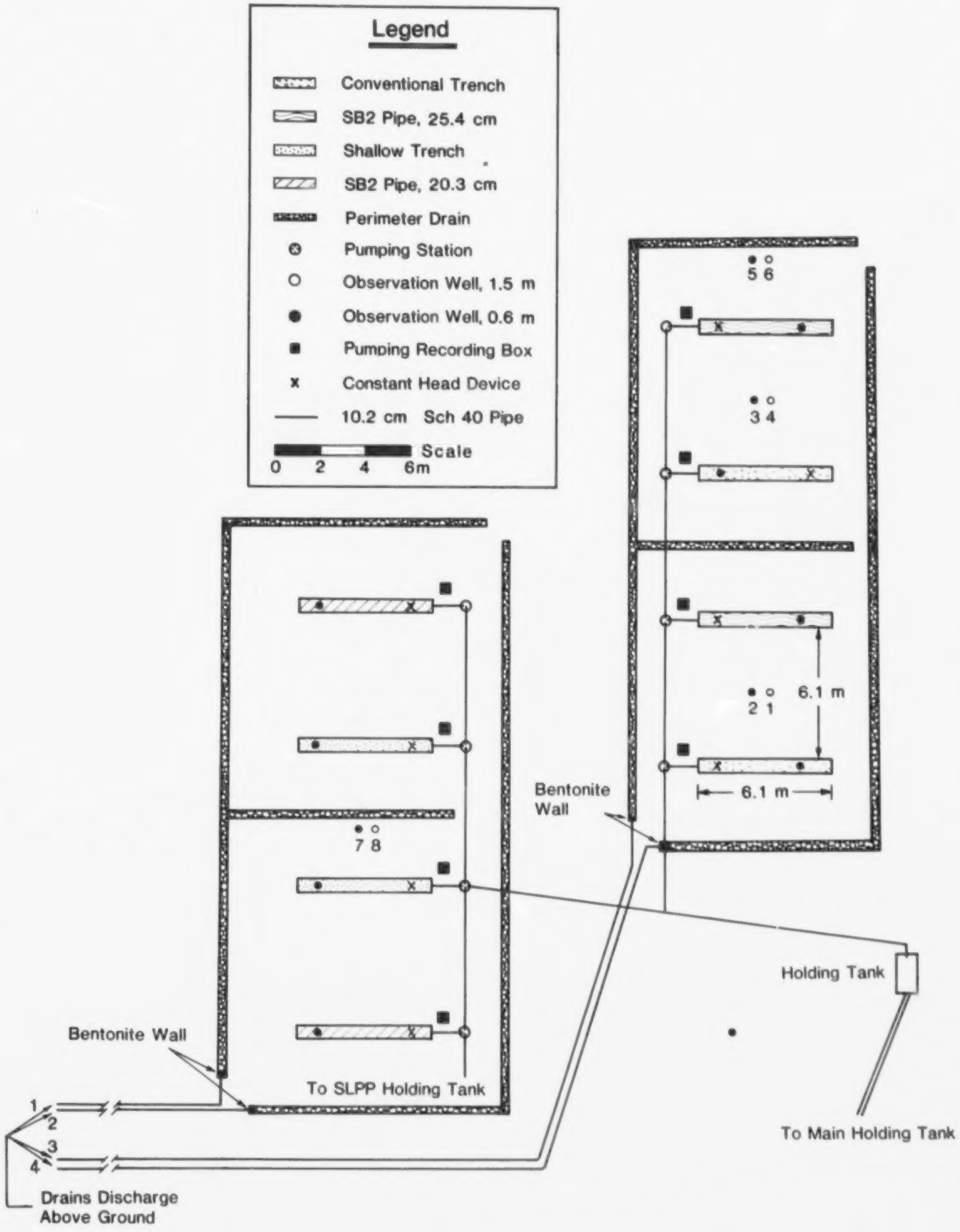

Legend
Conventional Trench
SB2 Pipe, 25.4 cm
Shallow Trench
SB2 Pipe, 20.3 cm
Perimeter Drain
Pumping Station
Observation Well, 1.5 m
Observation Well, 0.6 m
Pumping Recording Box
x Constant Head Device
10.2 cm Sch 40 Pipe
Scale
0 2 4 6m
5 6
3 4
2 1
7 8
6.1 m
6.1 m
Bentonite Wall
Bentonite Wall
To SLPP Holding Tank
Holding Tank
To Main Holding Tank
1
2
3
4
Drains Discharge Above Ground

Figure D-1. Overview of the various trench designs and perimeter drains.

45 cm (18 in) of soil was mounded above the trench to ensure against freezing and to prevent surface water from ponding above the trenches.

The gravelless trenches consisted of 25.4-cm (10-in) diameter corrugated plastic tubing covered with a porous wrap as manufactured by Advanced Drainage Systems, Inc. and marketed under the brand name of SB2® gravelless leach bed tubing. The recommendations of the manufacturer were followed when the fabric wrapped tubing was placed in the 50 cm (20 in) deep and 60 cm (24 in) wide trenches (Lang, 1981). The trenches were then backfilled in a manner similar to the backfilling of the conventional gravel-filled trenches.

In the fall of 1984, one year after the three original trench designs had been put into service, a fourth pair of trenches was installed. The 6.1 m (20 ft) long trenches, installed adjacent to the shallow trenches (Figure D-1), consisted of 20.3-cm (8-in) diameter corrugated SB2® plastic tubing. The SB2® tubing was installed in a manner similar to the installation of the 25.4-cm (10-in) SB2® tubing following the recommendations of Lang (1981), and the trenches were put into service shortly after their installation. Because the three original trench designs had been in service for one year prior to the installation of the 20.3-cm (8-in) diameter SB2® tubing, the reader is cautioned not to compare on a month-to-month basis the performance of the 20.3-cm (8-in) diameter SB2® with the original three pairs of trenches.

Effluent Collection and Application

Effluent was obtained by connecting the septic tanks of three nearby residences to a central storage tank. The effluent was then pumped from the central storage tank to a second storage tank where it flowed by gravity into pumping stations. Each trench had its own pumping station, consisting of a pump, check valve, gate valve, magnetic float switch, and time recording device (Figure D-3).

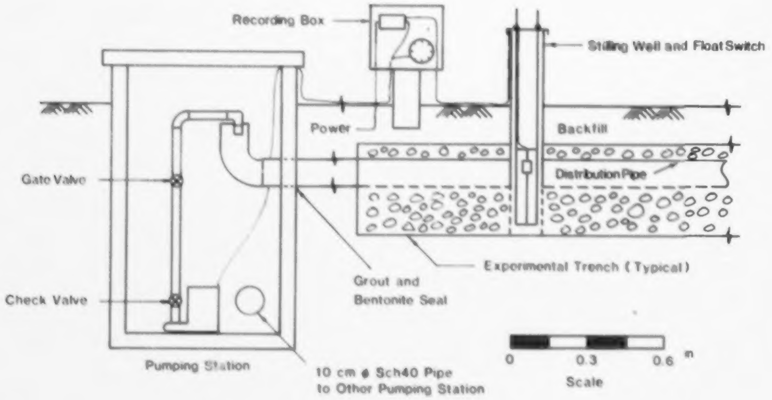

Recording Box
Stilling Well and Float Switch
Power
Backfill
Distribution Pipe
Gate Valve
Experimental Trench (Typical)
Grout and
Bentonite Seal
Check Valve
0
0.3
0.6
m
Scale
Pumping Station
10 cm ø Sch40 Pipe
to Other Pumping Station

Figure D-3. Cross section of effluent application and time recording instrumentation.

The mechanism used to apply effluent to each of the trenches effectively allowed for an accurate measurement of the volume of liquid absorbed by each trench. The mechanism consisted of a floating magnet that activated reed switches within the magnetic float switch (Figure D-4). When the depth of ponding in the trench fell to approximately 17 cm (7 in) as a result of effluent absorption, the magnetic float activated the lower reed switch, which in turn activated the pump, and effluent was applied to the trench. The application of effluent to the trench caused the magnetic float to rise, and when the depth of ponding reached a level of approximately 23 cm (9 in), the magnetic float activated a second reed switch causing the pump to turn off. The length of time the pump was running was recorded using digital timers. Each pump was individually calibrated in gallons per minute and the volume of effluent absorbed per day was calculated.

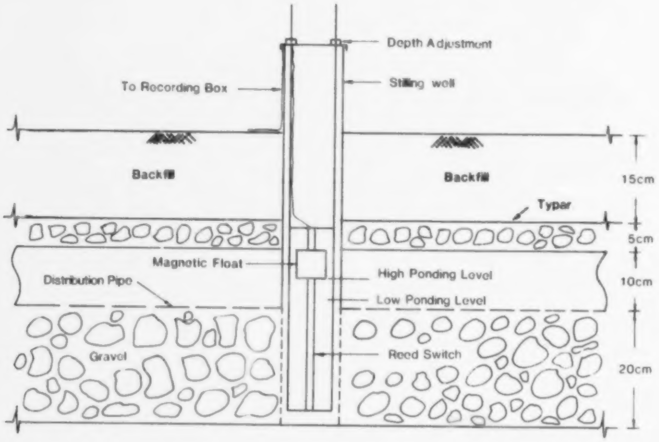

Depth Adjustment
To Recording Box
Stilling well
Backfill
Backfill
15cm
Typar
5cm
Magnetic Float
Distribution Pipe
High Ponding Level
Low Ponding Level
10cm
Gravel
Reed Switch
20cm

Figure D-4. Close-up of the magnetic float/reed switch that functioned as the constant head control device.

The switch pump timer mechanism allowed for septic tank effluent to be applied to the trenches with a relatively constant head of 18 to 23 cm (7 to 9 in). It also allowed for the application of effluent based on a trench's capacity to absorb it.

Septic Tank Effluent Sampling

Once or twice a week throughout the study, septic tank effluent grab samples were collected and analyzed for common wastewater constituents to determine the strength of the effluent.

Water Table Measurements

The distance from grade to the water table was measured once a week in observation wells located between the trenches as shown in Figure D-1.

Perimeter Drains

After a year of monitoring the trenches, it became apparent that a seasonally high water table was affecting their performance and limiting their capacity to absorb septic tank effluent. In an attempt to lower the water table, perimeter drains were installed around the trenches in the fall of 1984 (Figure D-1).

The perimeter drains, located approximately 3 m (10 ft) from the trenches, were 31 cm (1 ft) wide and 127 cm (50 in) deep and were constructed in the following manner. The trench was dug and a 5-cm (2-in) layer of 2.5-cm (1-in) diameter gravel was laid in the excavation. A 10-cm (4-in) diameter perforated pipe was placed above the gravel and the pipe was covered with additional gravel to a depth of 15 to 25 cm (6 to 10 in) from grade (Figure D-5). During the backfilling process, soil was mounded over the perimeter drains to prevent the development of low areas after settling occurred.

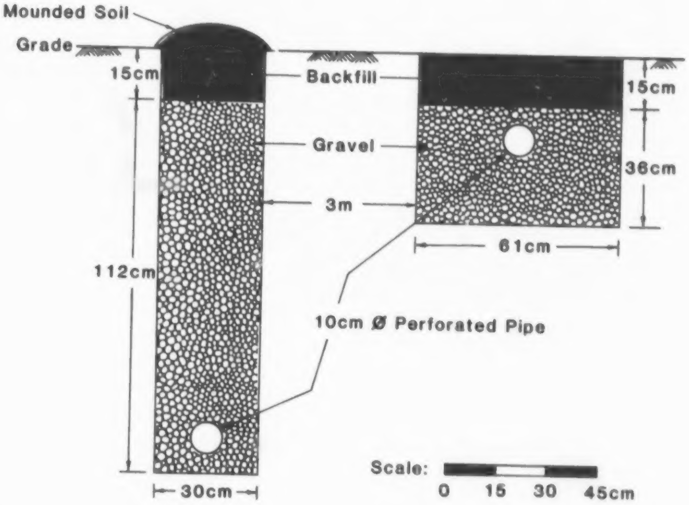


Figure D-5. Cross section of the perimeter drain installed next to the conventional gravel-filled trench.

Two pairs of perimeter drains were installed; the first pair was constructed around the conventional and SB2® (25.4-cm, 10-in diameter) trenches, and a second pair was constructed around the shallow trenches and the 20.3-cm (8-in) SB2® tubing. Each pair of drains was divided into two sections; one section diverted groundwater upslope from the trenches while the other collected water downslope and between the trenches as shown in Figure D-1. Each section of perimeter drain had a separate discharge pipe, which allowed for the collection of independent groundwater samples from each drain. This arrangement was selected so that it would be possible to determine individual discharge rates from each of the perimeter drains and to collect groundwater samples upslope and downslope from the trenches.

A limited groundwater sampling program was initiated when grab samples were collected from the discharging perimeter drains during the spring of 1985. Fecal coliform samples were collected in sterile glass containers while two other samples, collected in nonsterile cubitainers, were also obtained. One nonsterile sample was used for chemical analysis excluding chlorides, and the other sample was used for the analysis of chlorides. All of the samples were refrigerated within 1 hr after collection, and the samples used for the chemical analysis excluding chlorides were also fixed with 1 ml of concentrated H_2SO_4 per liter of sample.

Covering the Trenches with Black Plastic

In early July 1985, during the last 7 weeks of the study, black plastic was used to completely cover one each of the conventional, SB2®, and shallow trenches. The plastic, perforated to allow the rain to soak through it and into the soil beneath, extended 3 m (10 ft) on either side of the trench in order to kill any vegetation with roots that might have contributed to evapotranspiration. Within one week, all of the vegetation beneath the plastic had died. These three trenches remained covered with plastic until the end of August 1985 in an attempt to determine the impact of evapotranspiration on the trench's absorption rate. Because the plastic limited evaporation, and the death of the vegetation prevented transpiration, the predominant method of effluent disposal by the three covered trenches was soil absorption during the month of August 1985.

RESULTS AND DISCUSSION

Septic Tank Effluent Concentration

Results of the analysis of the septic tank effluent applied to the trenches (Table D-1) show that, in general, the strength of the effluent was slightly weaker than concentrations reported in the literature (Otis and Boyle, 1976).

TABLE D-1. CONCENTRATION OF SEPTIC TANK EFFLUENT APPLIED TO THE TRENCHES

Constituent	Mean	Range	Number of samples
	---------mg/l	-----------	
Chloride	81.9	19.8-250.0	73
COD	181.0	26.0-358.0	47
NH_3-N	23.9	2.3-36.9	47
NO_2+NO_3-N	0.1*	<0.1-0.5	47
TKN	28.9	1.9-45.1	46
Total-P	11.8	0.6-17.9	46

* The mean was calculated by assuming a value of 0.1 mg/l for samples having a concentration less than the detectable limit of 0.1 mg/l.

Absorption Rates

Because the original three trench designs (conventional, shallow, and 25.4-cm diameter SB2®) were installed and put into service one full year before the 20.3-cm (8-in) diameter SB2®, a direct comparison of the smaller diameter gravelless tubing with the original trenches should not be made. It is reasonable, however, to compare the performance of the original three pairs of trenches to each other, and to discuss the performance of the 20.3-cm (8-in) diameter SB2® separately.

The rate at which septic tank effluent was absorbed by each of the original six trenches was extremely variable from season to season, showing the effect of climatic conditions and other factors on trench performance (Figure D-6). In general, however, the conventional, 25.4-cm (10-in) SB2®, and the shallow trenches performed adequately during the summer months of July through September 1984, when the soil moisture content was low. During these three months, the conventional gravel filled trenches had an average absorption rate of 17 lpd per linear meter of trench, with the two other trench designs performing as well or better. The SB2® gravelless trenches had an average absorption rate of 24 lpd per linear meter of trench, while the shallow gravel filled trenches had an average absorption rate of 30 lpd per linear meter of trench.

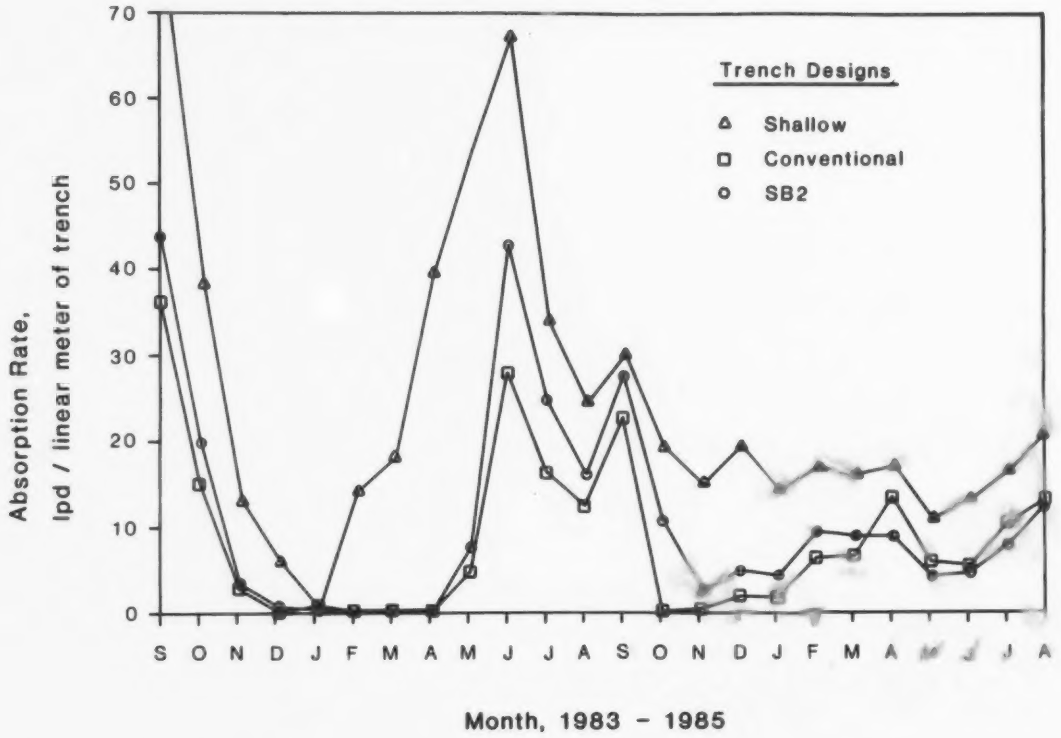


Figure D-6. Monthly average absorption rates for each trench design.

During the dry summer months there were differences in the absorptive capacities of the trenches, but any of the designs studied, if properly sized, would be acceptable during these months for the disposal of septic tank effluent in an Avonburg soil. Trench performance appears to be more critical during the wetter and colder seasons of the year when precipitation and soil moisture contents are high. It is during these wetter seasons that a comparison of the performance of the three trench designs is most meaningful.

During the cold and wet season from October 1983 through May 1984, the trenches were under a period of stress as shown by a significant deterioration in the absorption rates. Monthly average absorption rates were as low as 0 lpd per linear meter of trench for both the conventional and the SB2® gravelless designs. During the same 8-month period, on average, the shallow trenches performed significantly better (Figure D-6). However, during January 1984 when the water table was near the surface of the ground, the absorption rate of the shallow trenches was also near 0 lpd per linear meter of trench, indicating that none of the trench designs would be an effective year-round solution to the onsite wastewater disposal problems at this location unless other modifications were also used at the site.

The absorption rates of each trench design were grouped in three-month intervals to determine the effect of the season on trench performance. The results, summarized in Table D-2, show that, on average, there was little difference in the absorption rates of the conventional and SB2® trench designs throughout most of the study. During two of the nine seasons (Jul-Sep 1984 and Oct-Dec 1984), however, the absorption rate of the SB2® trenches was statistically greater than that of the conventional gravel filled trenches.

TABLE D-2. EXPERIMENTAL TRENCH ABSORPTION RATES DURING MULTI-MONTH TIME INTERVALS, (LPD PER LINEAR METER OF TRENCH)

	Trench design		
Time period	Conventional	25.4-cm SB2®	Shallow
Sep 1983	36*(A)†	43 (A)	73 (B)
Oct - Dec 1983	7 (A)	9 (A)	20 (B)
Jan - Mar 1984	0 (A)	0 (A)	15 (B)
Apr - Jun 1984	10 (A)	16 (A)	52 (B)
Jul - Sep 1984	17 (A)	24 (B)	30 (C)
Oct - Dec 1984	1 (A)	3 (B)	17 (C)
Jan - Mar 1985	6 (A)	8 (A)	16 (B)
Apr - Jun 1985	7 (A)	6 (A)	13 (B)
Jul - Aug 1985	12 (A)	10 (A)	19 (B)

* Average value of two trenches of the same design.

† Trench averages with the same letter for the same time period are not significantly different from each other.

The shallow trenches consistently performed better than the conventional or SB2® trenches throughout the study. Both the average monthly absorption rates (Figure D-6), as well as the average grouped seasonal rates (Table D-2), show that the shallow trenches had a significantly higher average absorption rate than the other two trench designs. While none of the trench designs evaluated were capable of disposing of septic effluent every month during the first year of the study, the shallow trenches had significantly fewer days of seasonal failure.

Performance of the 20.3-cm Diameter Gravelless Tubing

Under the conditions of this study, the 20.3-cm (8-in) diameter SB2® fabric wrapped tubing was capable of disposing of septic tank effluent at absorption rates similar to the 25.4-cm (10-in) diameter SB2® and the conventional gravel-filled trenches (Figure D-7). However, because the smaller diameter SB2® tubing was in service for only one year, a direct comparison between the absorption capabilities of the 20.3- cm (8-in) SB2® tubing and the other trenches should not be made.

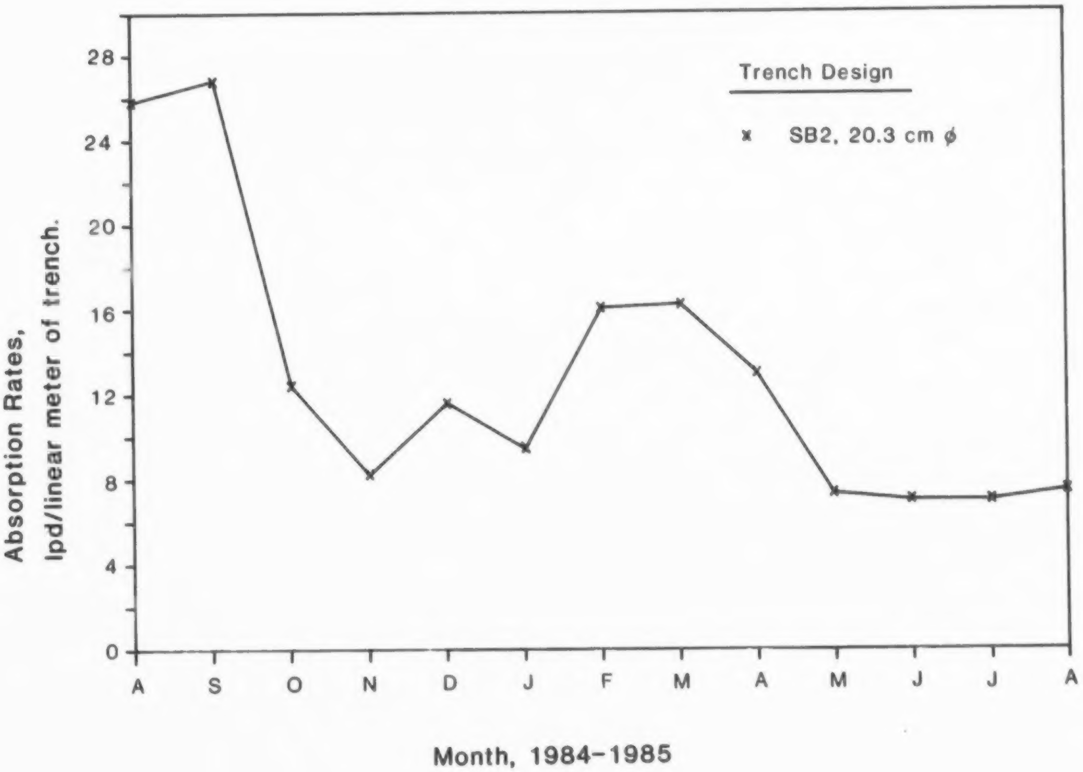


Figure D-7. Monthly average absorption rate of the 20.3-cm diameter SB2® tubing.

Effect of Perimeter Drains on the Water Table and Absorption

The absorption rates of the trenches were compared during the winter stress periods for the 2 years of this study to determine if the perimeter drains were effective in increasing the absorption capacity of the soil. These results are presented in Table D-3.

TABLE D-3. AVERAGE ABSORPTION RATES DURING TWO WINTER/SPRING STRESS PERIODS, (LPD PER LINEAR METER OF TRENCH)

Trench design	Stress period Oct 83-May 84	Stress period* Oct 84-May 85
Conventional	3† (A)§	5 (A)
Shallow	25 (A)	16 (B)
SB2®, 25.4 cm	4 (A)	6 (A)

* Perimeter drains were installed around the trenches in mid-October 1984.

† The average value for duplicate trenches of the same design is reported.

§ Trench averages with the same letter for the same trench design are not significantly different during the two stress periods.

There was no statistically significant increase in the absorption rates of the conventional or SB2® trenches during the 1984-1985 stress period as compared with the previous year. Throughout both of the stress periods, absorption rates were quite low, and the trenches could be considered to be at or near a point of seasonal failure. The installation of the perimeter drains did not appear to affect the soil moisture regime sufficiently to allow the year-round use of conventional gravel-filled or SB2® trenches at this site.

Unlike the conventional or SB2® trenches where there was no significant difference in the absorption rates before and after the installation of the perimeter drains, the absorption rate of the shallow trenches decreased significantly during the 1984-1985 stress period compared with the previous year. This decrease may be due to the fact that one of the duplicate shallow trenches had a relatively high absorption rate during the 1983-1984 stress period, resulting in a high average absorption rate for this trench design. During the 1984-1985 stress period the absorption rate of both of the shallow trenches was quite similar; neither of the two trenches outperformed the other, and as a result the average of the two trenches was lower than the average reported in the previous year. It is also possible that the shallow trenches were affected by the two 20.3-cm (8-in) diameter SB2® trenches that had been installed next to them. Regardless of the reason for a reduction in the average absorption rate of the shallow trenches during the 1984-1985 stress period as compared to the 1983-1984 stress period, the shallow trenches consistently absorbed septic tank effluent at a significantly higher rate than the conventional or 25.4-cm (10-in) diameter SB2® trenches throughout the 24-month monitoring program (Table D-2).

Chemical Composition of the Perimeter Drain Groundwater

Results of a limited perimeter drain groundwater sampling and analysis program are presented in Table D-4. The average concentration of chloride in the groundwater from the perimeter drains was greater than that found in groundwater obtained from a control well located in an area not influenced by the trenches. Averages ranged from 7.8 to 30.8 mg/l, indicating that groundwater from some of the perimeter drains was influenced by septic tank effluent moving through the soil. Average values for TKN and NO_2+NO_3-N were also greater than concentrations found in the control well. There was little difference in the groundwater obtained from the control well and the groundwater discharged from the perimeter drains for the constituents of NH_3-N and Total-P. Average fecal coliform numbers (MPN method) never exceeded 50/100 ml and the range of all of the samples never exceeded 200/100 ml, the standard for water contact for recreational use (Federal Water Pollution Control Administration, 1968). Under the conditions present in this study, a separation distance of 3 m (10 ft) between the trenches and the perimeter drains was adequate to keep highly concentrated septic tank effluent from entering the perimeter drains.

TABLE D-4. CHARACTERISTICS OF GROUNDWATER OBTAINED FROM THE DISCHARGING PERIMETER DRAINS

Parameter	Background well*	Drain 1	Drain 2	Drain 3	Drain 4
Chloride	7.2†	26.8	30.8	11.1	7.8
NH_3-N	0.4	0.4	0.5	0.1	0.2
NO_2+NO_3-N	0.6	1.9	2.7	1.9	1.8
TKN	1.1	3.1	2.1	1.8	1.5
Total-P	0.8	0.8	0.5	0.6	0.6
Fecal coliforms	<2	41.0	12.0	14.0	32.0
Max. recorded flow (lpm)	N/A	0.8	1.1	1.3	5.9

* 150-cm deep groundwater sampling well not affected by the trenches and used as a control (Effert and Cashell, 1985).

† Values are averages reported as mg/l, except fecal coliforms which are reported as the average number per 100 ml.

Formation of a Biological Mat

At the termination of the study the trenches were taken out of service and effluent was not applied to them for two weeks. After this two-week resting period, the trenches were dug up to determine the extent, if any, of the formation of a biological mat. Even though the trenches had been continuously inundated with septic tank effluent for more than two years, the gravel above the distribution pipe in the conventional and shallow trenches was not covered with a gelatinous black slime. There was a thin black slime layer on the gravel located beneath the pipe, but this was soon oxidized while the trenches were being excavated. There was no indication of the presence of a biological mat at the gravel/soil interface, but there was slime and other deposits inside the 10.4 cm (4 in) perforated distribution pipe. The trenches looked as if only a small amount of septic tank effluent had been applied to them.

Unlike the gravel-filled trenches, both the 20.3-cm (8-in) and 25.4-cm (10-in) SB2® trenches showed signs that septic tank effluent had been applied to them over an extended period. Effluent was present in the bottom of the corrugated tubing, and there was a noticeable buildup of gelatinous black slime. A well-developed black biological mat had also formed in the interface between the fabric and the soil, but the biomat extended only a few millimeters into the soil. There was no biological mat at the interface of the corrugations of the tubing and the fabric where the fabric pressed against the tubing. This gave the fabric a striped appearance. As reported by Anderson et al. (1983), the biomat was noticeably thicker at the bottom of the corrugated tubing than on the sides or top.

Absorption without the Assistance of Evapotranspiration

Throughout the study, the duplicate 25.4-cm (10-in) SB2® diameter trenches had absorption rates that were not significantly different from each other; they performed as identical duplicates. If the absorption rate of the SB2® trenches was due to evapotranspiration as well as soil absorption, a decrease in the measured absorption rate of the covered trench could be expected because the black plastic effectively reduced evapotranspiration. If evapotranspiration was significant, the absorption rate of the covered trench could be expected to decrease substantially, and the two trenches would no longer perform statistically as identical pairs.

During the month of August 1985, the duplicate 25.4-cm (10-in) diameter SB2® trenches continued to absorb septic tank effluent at statistically similar rates; they continued to operate as identical duplicates. The fact that there was not a significant reduction in the absorption rate of the covered SB2® trench compared with the uncovered trench of the same design suggests that either evapotranspiration did not play a significant role in the measured absorption rate during August 1985, or that the soils were capable of spreading the effluent out by capillary action beyond the plastic covering, utilizing a much larger evaporative area than expected.

SUMMARY AND CONCLUSIONS

The performances of three soil absorption field trench designs were compared during a two-year study. The trenches were installed in an Avonburg soil known to have severe site limitations for the installation of onsite wastewater treatment systems. The limitations included impermeable soils, shallow depth to an impermeable layer, and a seasonally high water table.

Results show that all three of the trench designs can be used in an Avonburg soil to absorb and dispose of septic tank effluent during the summer months when absorption rates are high. During the summer, the shallow trenches absorbed on average, 30 lpd per linear meter of trench, while the SB2® and conventional gravel-filled trenches absorbed 24 and 17 lpd/m of trench, respectively. During the winter and spring seasons, however, the conventional and SB2® trenches absorbed little or no effluent and they were in a state of seasonal failure for months at a time.

A comparison of the conventional gravel-filled trench and the SB2® corrugated fabric covered tubing shows that during 75% of the two-year study, there was no statistically significant difference in the absorption rate of the conventional 60-cm (2-ft) wide gravel-filled trenches vs. the SB2® gravelless trenches.

The shallow trenches had a significantly higher absorption rate than the conventional or SB2® trenches. However, the shallow trenches also experienced periods when the absorption rates were so low that the trenches were in a state of seasonal failure. Therefore, without modification to the site, an Avonburg soil with similar limitations is not suitable for the year round absorption and disposal of septic tank effluent using the trench designs evaluated in this study.

There was no statistically significant increase in the absorption rates of the trenches following the installation of the perimeter drains. A separation distance of 3 m (10 ft) between the trenches and the perimeter drains was adequate to keep highly concentrated septic tank effluent from entering the drains.

During the summer, absorption of the septic tank effluent into the soil appears to be the predominant method of effluent disposal. At least with respect to the 25.4-cm (10-in) SB2® gravelless tubing, evapotranspiration did not play a significant role in effluent absorption, and the data do not suggest that the effect of evapotranspiration on the absorption rate was any greater for the other two trench designs.

Even though the trenches had been continuously inundated with septic tank effluent for two years, a biological mat had not developed at the gravel/soil interface of the conventional and shallow trenches, or if it was present it had been oxidized during a two-week resting period. The thin layer of black gelatinous material on some of the gravel was quickly oxidized during the excavation of the trench. The SB2® trenches did have a characteristic biological mat around them, and gelatinous slime was evident in the corrugated tubing.

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Appendix E

EFFECT OF DOSING FREQUENCY ON SIMULATED MOUND SOIL ABSORPTION SYSTEMS*

John Clark

INTRODUCTION

A mound soil absorption system, an alternative method of onsite wastewater disposal, is elevated above the soil by using a suitable sand fill material. This keeps the seepage beds, which are located within the sand, above limiting site conditions. Septic tank effluent is typically applied to the seepage beds under pressure, and treatment of the effluent occurs when it passes through the sand and plowed layer of topsoil on which the mound is built. Ultimately, the liquid is absorbed by the upper horizons of soil, and because it has been treated by the sand and soil, mounds can be used in areas with site limitations, including a high water table, creviced bedrock, or shallow soil depth to bedrock.

Mounds provide various degrees of treatment depending on the type of sand fill that is used, the soil type beneath the mound, the hydraulic loading rate, and dosing frequency. The dosing frequency is dependent on water usage within the home, and typically averages four times per day. This study was undertaken to determine the effect of timed dosing frequencies on the treatment of domestic septic tank effluent passing through sand and soil columns that were used to simulate a mound.

MATERIALS AND METHODS

Three sand/soil columns were constructed in 15-cm ID by 1.8-m long Plexiglass cylinders. The columns contained four distinct zones approximating those found in a mound: an insulating soil cover, a gravel bed, sand fill, and a plowed topsoil layer over which a mound is typically constructed (Figure E-1).

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* Extended abstract from M.S. thesis. Department of Civil and Environmental Engineering, University of Cincinnati, 1980.

The top zone of each column consisted of 30 cm of topsoil, which in an actual mound promotes the runoff of rainwater and insulates the sand fill. Typically, grass is grown on this area, but in this study, no attempt was made to grow grass or to simulate rainfall.

The second zone consisted of 30 cm of 1.3- to 1.9-cm nominal diameter gravel representing the distribution trench or bed of a mound. A 1.3-cm diameter Plexiglass cylinder was inserted through the wall of each column at a depth of 10 cm in the gravel and served as the waste distribution pipe. The gravel was separated from the overlying topsoil with a layer of coarse paper to prevent topsoil from filling the void spaces within the gravel zone.

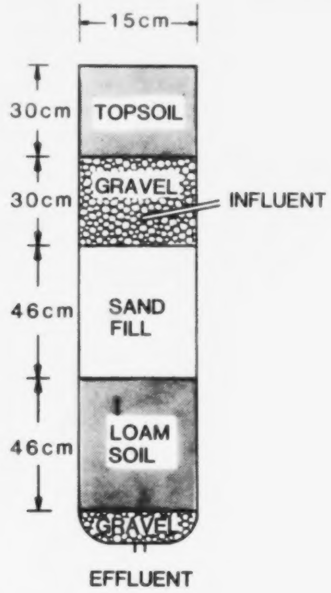


Figure E-1. Cross section of the sand/soil column used to simulate a mound.

The third zone represented the sand fill. This zone consisted of 46 cm of medium grade sand corresponding to Ohio D.O.T. designation 703.03 (masonry sand) and it complied with recommended guidelines for mounds (Standard Slag Co., 1977). Five 0.8-cm diameter holes were drilled through the column walls at depths of 12 and 20 cm below the gravel/sand interface to allow lateral gas movement in the sand fill as recommended by Magdoff et al. (1974). Nylon elbows were inserted in the holes and Tygon® tubing was extended upward from the elbows to prevent loss of flow from the holes following dosing events.

The fourth zone of each column consisted of 46 cm of soil representing the basal area beneath a mound. The soil used was a silt loam as determined by a particle size analysis, and was classified as an Eldean loam (Lerch et al., 1982). Before the soil was added to the column it was ground to pass a No. 4 sieve with fines passing a No. 80 sieve. The top 15 cm of this fourth zone was placed in the columns uncompacted to simulate the plowed native soil surface. In a mound system, the soil surface is plowed before construction to help promote infiltration of the treated wastewater into the soil. The lower 30 cm of soil was packed to a density of 8543 kg/cu m to represent the undisturbed soil. A layer of 2.5 to 5.0 cm of 0.1-cm nominal diameter gravel was placed below the soil to prevent the loss of soil and provide drainage during effluent collection.

Domestic septic tank effluent was collected twice weekly from a family residence and was stored at 4°C. The effluent was warmed for approximately 24 hr at 20°C before dosing to the sand/soil columns. Dosing was accomplished using a timer and pump for each column. Each column was dosed at a rate of 10.2 lpd/sq m. However, the dosing frequency was varied for each column. One column was dosed at 3-hr intervals, the second at 6-hr intervals, and the third at 12-hr intervals. Column effluent samples were collected by gravity from each column and stored in a cooler at 4° C. The columns began operation on February 9, 1982. Samples for analysis were collected from June 18 through November 4, 1982.

Two sets of samples were collected twice weekly. Each sample set consisted of four samples: influent (septic tank effluent), 3-hr column effluent, 6-hr column effluent, and 12-hr column effluent. One set of samples was analyzed immediately for biochemical oxygen demand (BOD_5), total suspended solids (TSS), pH, fecal coliforms, and orthophosphate (Ortho-P). The second set was preserved with concentrated H_2SO_4 and sent to the U.S. Environmental Protection Agency, Cincinnati, Ohio, for analysis for total Kjeldahl nitrogen (TKN), ammonia (NH_3-N), nitrite (NO_2-N), nitrate (NO_3-N), chemical oxygen demand (COD), and total phosphorus (Total-P). The methods used for analysis are in Standard Methods for the Examination of Water and Wastewater, 15th edition (1980) and Methods for Chemical Analysis of Water and Waste (1979).

RESULTS AND DISCUSSION

The data indicate that statistically significant treatment occurred for all measured parameters at all three dosing frequencies (Table E-1). The average values for the measured parameters and the 0.05 level of significance were used for the statistical analysis. The following summarizes the statistical analysis and interpretation of results.

Nitrogen: Influent nitrogen species were typical for septic tank effluent with 83% NH_3-N, 17% Organic-N, and less than 1% NO_2+NO_3-N. The effluent data indicate that the Kjeldahl nitrogen was nitrified during passage through the columns, with the 3-hr cycle providing 95% nitrification, the 6-hr cycle providing 92% nitrification, and the 12-hr cycle providing 85% nitrification. Total nitrogen removal during passage through the columns was 52% for the 3-hr cycle, 36% for the 6-hr cycle, and 21% for the 12-hr cycle. Biological denitrification was probably responsible for the nitrogen removal.

Phosphorus: Nearly complete removal of phosphorus occurred in all three columns with no apparent difference attributable to the dosing frequencies. Total phosphorus removal was 98 to 99% and orthophosphate removal was greater than 99%.

Biochemical Oxygen Demand: The low hydraulic loading rate allowed high removals of BOD_5 with no apparent difference resulting from the dosing frequencies. BOD_5 removal was estimated to be 93% for all three loading rates.

Chemical Oxygen Demand: COD was reduced by 90 to 91% in the columns, with no obvious difference attributable to dosing frequency.

TABLE E-1. MEAN VALUES FOR COLUMN INFLUENT AND EFFLUENT, MG/L

Parameter	Septic tank effluent	3-hr eff.	6-hr eff.	12-hr eff.
TKN	150.0	7.6 (95)*	12.1 (92)	22.5 (85)
NH_3-N	124.0	7.2 (94)	11.9 (90)	22.5 (82)
Organic Kjeldahl nitrogen	26.0	0.5 (98)	0.2 (99)	0.1 (99)
NO_2-N	0.2	0.2 (0)	0.2 (0)	0.2 (0)
NO_3-N	0.3	64.0 (I)†	84.0 (I)†	96.0 (I)†
Total-N	150.0	72.0 (52)	96.0 (36)	119.0 (21)
Total-P	19.5	0.3 (98)	0.2 (99)	0.2 (99)
Ortho-P	20.6	0.1§ (99)	0.1§ (99)	0.1§ (99)
BOD_5	306.0	<20§ (93)	<20§ (93)	<20§ (93)
COD	578.0	57.0 (90)	52.0 (91)	59.0 (90)
Fecal coliforms	5.7x10^5	<2§ (99)	<2§ (99)	<2§ (99)
TSS	144.0	0§ (99)	0§ (99)	0§ (99)
pH**	6.5-8.1	7.3-8.1	7.4-8.0	7.5-8.1

* Percent reduction.

† An increase occurred.

§ Values were usually less than the detectable limit, therefore means are estimates.

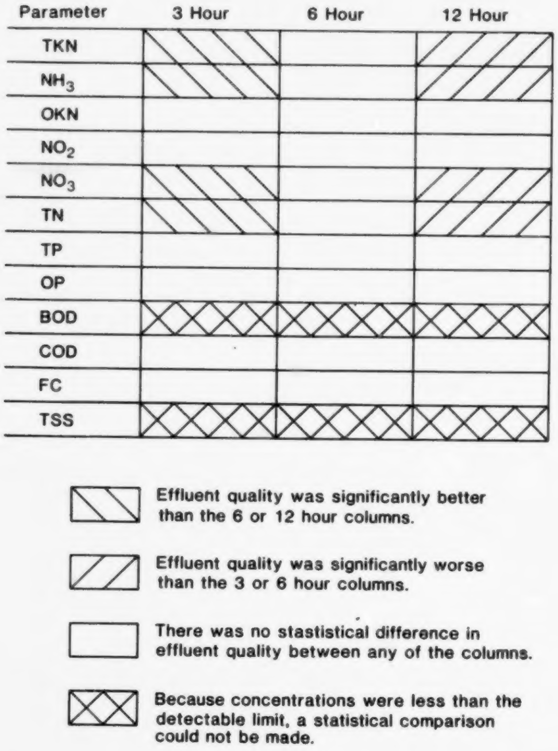
** Range of pH.

Fecal Coliforms: No positive tubes were produced during the completed test of the multiple tube fermentation technique indicating that all three dosing frequencies produced effluents with less than 2 fecal coliforms/100 ml.

Total Suspended Solids: As expected, TSS were removed by filtration in the sand and/or soil of the columns. Total suspended solids concentrations were less than the detectable limit in all of the column samples, and as a result no acceptable data were obtained.

Formation of a Biological Mat: The formation of a biological mat was not apparent in any of the columns at any time during the study. However, after the columns had been in service for approximately 8 months, a greying of the gravel occurred.

The data show that the dosing frequency did not affect the treatment of the septic tank effluent in the sand/soil columns, with the exception of nitrogen removal. Nitrogen removal was greatest with the 3-hr dosing frequency and least with the 12-hr dosing frequency (Figure E-2). It is hypothesized that the dosing frequency may have affected the treatment by causing different flow regimes within the columns. In the 3-hr column, the small frequent doses produced unsaturated flow in the sand. Under conditions of unsaturated flow, the septic tank effluent percolates slowly through the sand as a surface film on the sand particles, allowing aerobic organisms to nitrify the NH_3-N and organic Kjeldahl nitrogen to NO_3-N. The unsaturated flow also allows continuous diffusion of oxygen into the sand.



Parameter	3 Hour	6 Hour	12 Hour
TKN	▧		▨
NH_3	▧		▨
OKN			
NO_2			
NO_3	▧		▨
TN	▧		▨
TP			
OP			
BOD	▩	▩	▩
COD			
FC			
TSS	▩	▩	▩

▧ Effluent quality was significantly better than the 6 or 12 hour columns.

▨ Effluent quality was significantly worse than the 3 or 6 hour columns.

□ There was no stastistical difference in effluent quality between any of the columns.

▩ Because concentrations were less than the detectable limit, a statistical comparison could not be made.

Figure E-2. Significance of the quality of effluent produced by the three sand/soil columns.

In contrast, observation of the 12-hr dosing event indicated that sufficient wastewater may have been applied to produce saturated flow in the sand. This flow regime allowed the septic tank effluent to move more rapidly through the sand than unsaturated flow, and therefore reduces the time available for contact with nitrifying organisms. The rapid movement of septic tank effluent through the sand also displaces gases, including oxygen, and thereby reduces the amount of oxygen available for nitrification.

Denitrification can also be affected by the flow regime. Denitrification is expected to occur in an anaerobic (saturated) zone near the sand/soil interface caused by the lower permeability of the soil. The slow percolation of the septic tank effluent through the 3-hr column may have allowed the depth of this saturated zone to remain essentially constant. In the 12-hr column, the septic tank effluent percolated rapidly through the sand. This rapid percolation temporarily increased the depth of the saturated zone, and the head at the sand/soil interface may temporarily increase the rate of infiltration of flow into the soil. As the flow infiltrates into the soil, the depth of the saturated zone decreases. This flow regime may have produced a continuously saturated zone with less depth than the continuously saturated zone in the 3-hr column. The result would be a decrease in the time of contact of the nitrified waste with denitrifying microorganisms in the saturated zone.

CONCLUSIONS

In this study, when septic tank effluent was applied to sand/soil columns at a rate of 10.2 lpd/sq m, and at time intervals of 3, 6, and 12 hours, all three columns provided a significant level of treatment of the effluent for the parameters of total Kjeldahl nitrogen, ammonia, organic Kjeldahl nitrogen, total nitrogen, total phosphorus, orthophosphate, BOD_5, COD, fecal coliforms, and total suspended solids. There was not a significant change in nitrite concentration between the septic tank effluent and the sand/soil column effluents. The nitrate concentration significantly increased as the septic tank effluent passed through the columns. The pH also increased slightly.

Comparison of the sand/soil column effluents indicated that the column that was dosed every 3 hours produced an effluent with significantly lower total Kjeldahl nitrogen, ammonia, nitrate and total nitrogen than did the columns dosed every 6 and 12 hours. The 12-hr column produced an effluent with significantly higher total Kjeldahl nitrogen, ammonia, nitrate, and total nitrogen than did the 3- or 6-hr columns. There was not a significant difference in organic Kjeldahl nitrogen, nitrite, COD, total phosphorus, orthophosphate, or fecal coliforms between the three sand/soil column effluents. The pH of the three sand/soil column effluents had an equivalent range of values. Because most of the samples produced measurements less than the detectable limit, BOD_5 and total suspended solids could not be statistically compared.

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Appendix F

A COMPARISON OF THE PERFORMANCE OF TWO ONSITE INTERMITTENT SAND FILTERS*

David Effert and Margaret Cashell

INTRODUCTION

Soil conditions or other geological factors render many areas of the United States unsuitable for the installation of a conventional septic tank/soil absorption field system for onsite waste disposal. Systems that do not rely on the soil for final treatment and disposal of the wastewater are therefore often installed in these areas. Some of these systems discharge wastewater directly to a stream or other watercourse for final effluent disposal. Nuisance conditions or serious health problems could result from this if the wastewater is only partially treated. To reduce such potential problems, intermittent sand filters have been used to treat effluent from systems such as septic tanks and home aerobic units prior to discharge to a watercourse.

Intermittent sand filters have been used to polish wastewater that has been pretreated by a septic tank or home aerobic unit. The filters are relatively small, resulting in limited material and construction costs, they produce a high quality effluent that meets or exceeds surface discharge requirements in many states, and they can be easily maintained or rehabilitated, if necessary.

Although intermittent sand filters have been shown to be an effective and reliable method of polishing pretreated wastewater, little research has been done to determine the extent of pretreatment necessary for a filter to produce a high quality effluent. The primary objective of this study, therefore, was to determine whether different levels of wastewater pretreatment, as provided by a septic tank or home aerobic treatment unit, would have an effect on the performance and reliability of intermittent sand filters. The secondary objectives were to determine the operation and maintenance (O&M) requirements of intermittent sand filters when they are used by homeowners over an extended period of time, and to evaluate construction wastewater and O&M costs.

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* In: Fourth Ohio Environmental Engineering Conference. Amer. Soc. of Civil Engrs. Columbus, Ohio. 1984.

MATERIALS AND METHODS

Two private residences in Hamilton County, Ohio, were selected for the construction of the intermittent sand filter systems. At each location a fully operational pretreatment system, either a 4542-l (1200-gal) septic tank or a 1892-lpd (500-gpd) Jet, Inc., extended aeration package plant, was installed along with the sand filter. Construction took place during the winter of 1981 and the spring of 1982.

Filter Design

The sand filters at both sites were designed and installed using similar materials and construction practices. The basin for each filter consisted of a 2.44 m (8 ft) by 3.05 m (10 ft) concrete structure, which was 1.68 m (5.5 ft) deep (Figure F-1). The media for each filter consisted of 76 cm (30 in) of washed sand that had an effective size of 0.4 mm (0.016 in) and a uniformity coefficient of 2.5. Directly beneath the sand was a 10-cm (4-in) layer of pea gravel that overlaid 28 cm (11 in) of 2.5-cm (1-in) nominal diameter gravel. A 10-cm (4-in) diameter perforated pipe, vented above the sand, was used to collect the wastewater in the underdrain. Reinforced and insulated plywood was used to cover the filter to preclude any odor problems in the area of the filter and to prevent the filters from freezing. The covers were bolted to the basin to prevent any unauthorized entry and to satisfy local zoning laws.

A submersible pump, activated by a mercury float switch, dosed the pretreated wastewater onto the filter through a 5.1-cm (2-in) diameter elbow and tee. A splash plate was placed beneath the elbow and tee to prevent erosion of the sand during periods when the effluent was being applied as shown in Figure F-2.

Based on recommended loading rates (USEPA, 1980), the intermittent sand filters were designed to operate at a maximum hydraulic loading rate of 162 lpd/m^2 (4.0 gpd ft^2) and 326 lpd m^2 (8.0 gpd ft^2) for septic tank effluent and aerobic unit effluent, respectively. These loading rates were selected because it was the intent of this study to evaluate the performance of each filter under typical design and operational conditions. The actual loading rate, dosing frequency and other design criteria are presented in Table F-1.

Filter Operation

Water usage at the residence where the septic tank pretreated the wastewater was relatively constant over the period of the study. From December 1981, when the filter system was put into service, until September 1983, water usage averaged very close to 946 lpd (250 gpd) for each of the six three-month water billing periods, resulting in an average filter hydraulic loading rate of 150 lpd/m^2 (3.7 gpd/ft^2).

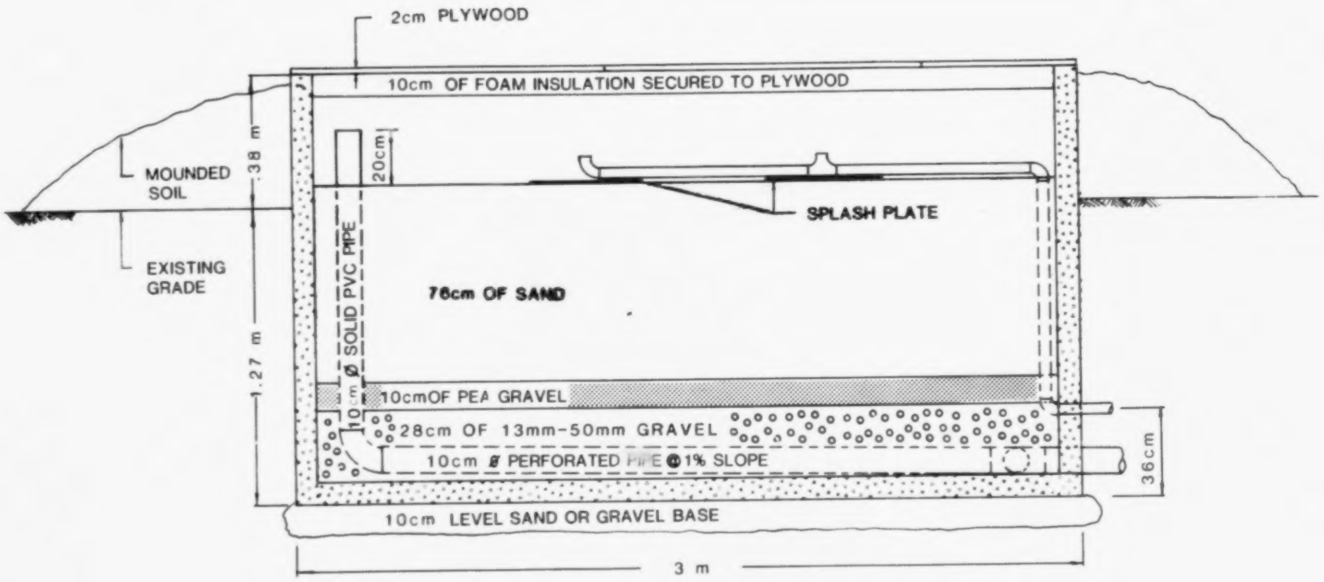
2cm PLYWOOD
10cm OF FOAM INSULATION SECURED TO PLYWOOD
MOUNDED SOIL
EXISTING GRADE
.38 m
1.27 m
20cm
10cm Ø SOLID PVC PIPE
SPLASH PLATE
76cm OF SAND
10cmOF PEA GRAVEL
28cm OF 13mm-50mm GRAVEL
10cm Ø PERFORATED PIPE @ 1% SLOPE
36cm
10cm LEVEL SAND OR GRAVEL BASE
3 m

Figure F-1. Sand filter cross-sectional view.

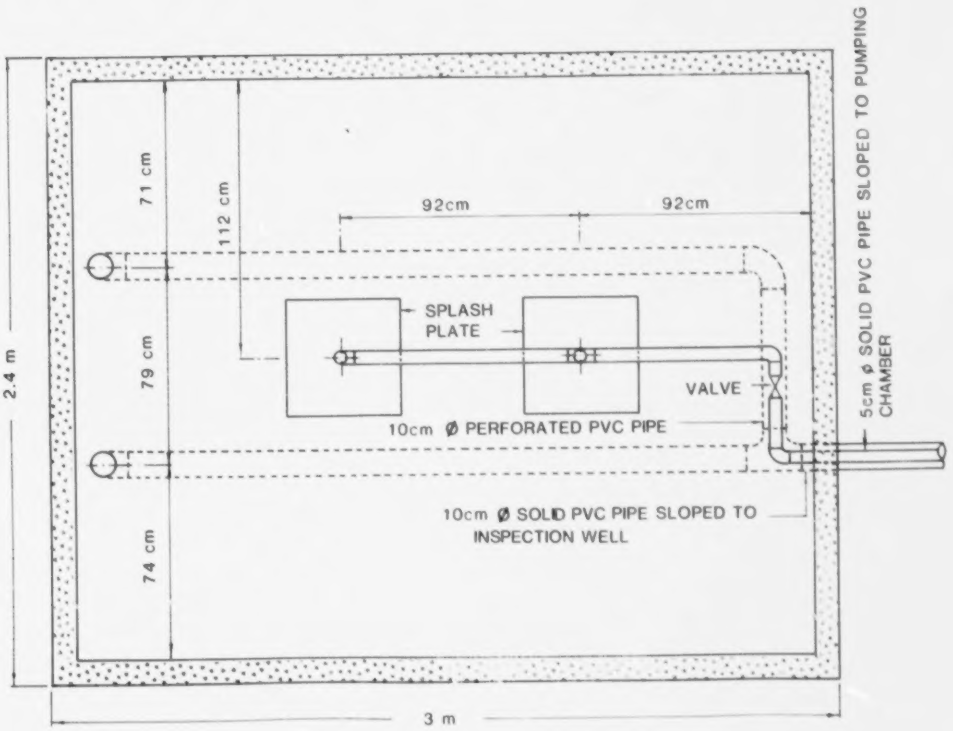
71 cm
112 cm
92cm
92cm
SPLASH PLATE
79 cm
2.4 m
VALVE
10cm ø PERFORATED PVC PIPE
5cm ø SOLID PVC PIPE SLOPED TO PUMPING CHAMBER
10cm ø SOLID PVC PIPE SLOPED TO INSPECTION WELL
74 cm
3 m

Figure F-2. Sand filter plan view.

Water usage at the residence where the aerobic unit pretreated the wastewater was not consistent over the period of the study. During the first 240 days the filter was in service, beginning in March 1982, water usage averaged 1938 lpd (512 gpd) resulting in an average sand filter hydraulic loading rate of 305 lpd/m^2 (7.5 gpd/ft^2). Water usage was high during this time because leaking plumbing caused an excessive volume of wastewater to be generated. Even with the leaking plumbing and the extra wastewater that resulted, the average volume of pretreated wastewater applied to the filter was less than the filter's design hydraulic loading rate.

In late September 1982, the residence was put up for sale and left unoccupied for one month. During that time, the sand filter was taken out of service and rested, and the leak in the plumbing was repaired. The new occupants used considerably less water than the previous occupants, and from October 1982, to August 1983, the sand filter received an average of 303 l (80 gal) of pretreated wastewater per day for an average hydraulic loading rate of 49 lpd/m^2 (1.2 gpd/ft^2).

TABLE F-1. INTERMITTENT SAND FILTER DESIGN AND LOADING RATE DATA

Item	Preliminary treatment provided by Septic tank	Jet aerobic unit
Period of operation	Dec 1981-Sep 1983	Mar 1982-Sep 1983
Size of sand filter	2.2 m x 2.8 m	2.2 m x 2.8 m
Surface area of filter	6.3 m^2	6.3 m^2
Depth of media:		
Sand	76 cm	76 cm
Pea gravel	10 cm	10 cm
Gravel	28 cm	28 cm
Characteristics of sand:		
Effective size	0.44 mm	0.44 mm
Uniformity coefficient	2.5	2.5
Design hydraulic loading rate	162 lpd/m^2	326 lpd/m^2
Actual hydraulic loading rate	150 lpd/m^2	305 lpd/m^2* 49 lpd/m^2†
Dosing quantity (volume/dose)	283 l	254 l
Dosing frequency (doses/day)	3.3	7.6* 1.2†

* Mar 1982 to Sep 1982.
† Oct 1982 to Sep 1983.

Sampling and Analysis

Influent and effluent composite samples from each sand filter were collected twice a week for an 18-month period beginning in April 1982. Influent samples were collected over the period of a dosing event by tapping into the manifold between the submersible pump and the distribution pipe, and connecting a section of Tygon® tubing to the tap. When the pump was turned on, a portion of the wastewater was diverted through Tygon® tubing to a chilled container for storage. An automatic sampler, activated by a float switch, was used to collect the sand filter effluent composite sample. The effluent passed through a sampling well prior to its discharge. When the level of the effluent rose in the well, a float switch also rose, turning on an automatic sampler. The sampler then pumped effluent out of the well and into a sample bottle. When the filter stopped discharging, the effluent drained from the sampling well via a small weep hole, and the float switch dropped down causing the automatic sampler to purge the Tygon® tubing, and turn itself off. Typically, two dosing events were sampled during a 24-hr period. The samples were kept on ice or refrigerated until the analyses were performed.

The analyses for fecal coliforms, total suspended solids, pH, dissolved oxygen (D.O.), and biochemical oxygen demand (BOD_5) were performed by technicians in the Department of Civil and Environmental Engineering at the University of Cincinnati. The analyses for chemical oxygen demand (COD), ammonia, total Kjeldahl nitrogen, nitrate, nitrite, and total phosphorus were conducted by the United States Environmental Protection Agency's Environmental Monitoring and Support Laboratory, Cincinnati, Ohio. The analyses were conducted according to either Standard Methods for the Examination of Water and Wastewater (American Public Health Association, 1980) or Methods for Chemical Analysis of Water and Wastes (USEPA, 1979).

RESULTS AND DISCUSSION

Data Analysis

Prior to conducting a statistical analysis of the data, it was observed that a few of the data points appeared to be extreme values, either high or low, which were not representative of the actual performance of the wastewater treatment systems studied. These outliers could have resulted from sampling or laboratory error. In order to identify the outliers and exclude them from the raw data, the Dixon Extreme-Value Test was used (Dixon, 1951). This statistical test can be used to calculate the maximum ratio of the differences between extreme-ranking observations to be expected at various probability levels for a given set of data. When this test was applied to the raw data and the difference between the ratio of the actual extreme ranking data points exceeded the expected maximum ratio at the 95% probability level, the data point was presumed to be an outlier, and it was rejected from the data set. Less than 0.75% of the data points were determined to be outliers at the 95% probability level.

After the outlier data points were rejected, a statistical analysis of the data was performed. The calculations for each parameter include the mean, the 95% confidence limits around the mean, the range of values, the number of samples analyzed, and the coefficient of variation. The coefficient of variation is a dimensionless value calculated by dividing the standard deviation by the mean. The lower the number, the less variable the samples for a given parameter.

The 95% confidence interval around the mean was calculated using the statistical formula usually applied to a normally distributed population. The tests of skewness and kurtosis determined that not all of the parameters were normally distributed, and if the parameter was not normally distributed, the Central Limit Theorem was applied when calculating the confidence interval. This theorem states that for large samples, the sampling distribution of the mean is approximately normally distributed regardless of how the parent population is distributed (Daniel, 1978). When the Central Limit Theorem is applied, the formula used to calculate the confidence interval is the same for both normally and non-normally distributed populations. In this instance, the 95% confidence interval around the mean was calculated as the mean ±1.96 X (standard error).

Effluent Quality

Septic Tank/Sand Filter System--

During the 18-month sampling period, from April 1982 to September 1983, the sand filter performed extremely well with respect to BOD_5 removal (Table F-2). Biochemical oxygen demand concentrations were reduced an average of 96% through the filter, resulting in an average effluent concentration of 4 mg/l. The Ohio Administrative Code standard for the discharging of treated wastewater off of private property is a mean BOD_5 of 20 mg/l (Ohio Department of Health, 1977). One hundred percent of the 68 sand filter effluent samples analyzed met the standard.

COD concentrations were reduced 75% through the sand filter, to an average of 76 mg/l. However, the values range from 4 to 463 mg/l, and the large coefficient of variation could indicate that unlike BOD_5, the filter may not provide a consistent level of removal of COD.

Suspended solids were reduced an average of 68% through the filter, for an average effluent concentration of 17 mg/l. Other researchers have shown similar reductions (Sauer, 1975). However, a direct comparison of this value with other reported intermittent sand filter effluent data is difficult, because the type and size of the sand, its coefficient of variability, the hydraulic and organic loading rates, the frequency and duration of the dosing event, and the age of the filter will affect filter performance and effluent quality.

The Ohio Administrative Code standard for the discharging of treated wastewater off of private property is a mean suspended solids concentration of 40 mg/l (Ohio Department of Health, 1977). Eighty-nine percent of the 73 sand filter effluent samples collected met this standard.

The septic tank effluent contained a geometric mean of 2.2×10^5 fecal coliforms per 100 ml of sample, and is characteristic of septic tank effluent (Effert, 1977). The filter reduced the fecal coliforms by approximately two logs to a geometric mean of 1.6×10^3 per 100 ml, but surface water standards for water contact recreation use are a geometric mean of 200 fecal coliforms/100 ml (Federal Water Pollution Control Administration, 1968). Given the hydraulic loading rates and dosing frequency reported in the study, disinfection of the sand filter effluent would be necessary in order to achieve the surface water standard.

Dissolved oxygen and pH measurements show that the sand filter's effluent achieved recommended standards for fish survival. While the dissolved oxygen concentration of the septic tank effluent was not measured in this study (it is assumed to be negligible because of the anaerobic conditions of the septic tank), the dissolved oxygen concentration of the sand filter effluent averaged 6.7 mg/l, and was never less than 4.0 mg/l.

This level is acceptable for wildlife use, and for the propagation of both warm and cold species of freshwater fish (Hammer, 1975). The median pH of the wastewater increased 0.3 units through the filter to 7.5. Nevertheless, 100% of the 83 sand filter effluent samples tested for pH were within the safe range for fish protection of 6.5 to 8.5 (Hammer, 1975).

TABLE F-2. EFFLUENT QUALITY DATA, SEPTIC TANK/SAND FILTER SYSTEM

Parameter	Septic tank effluent Apr 1982-Sep 1983	Sand filter effluent Apr 1982-Sep 1983*
BOD_5, mg/l		
Mean	127	4
95% Conf. int.	117-137	4-6
Range	10-285	2-13
No. of samples	109	68
Coef. of var.	0.43	0.94
COD, mg/l		
Mean	310	76
95% Conf. int.	291-329	55-98
Range	21-571	4-463
No. of samples	104	75
Coef. of var.	0.31	1.23
Suspended solids, mg/l		
Mean	53	17
95% Conf. int.	49-57	11-23
Range	20-132	1-130
No. of samples	111	73
Coef. of var.	0.36	1.47
Fecal coliforms, No. per 100 ml		
Geometric mean	2.19×10^5	1.62×10^3
95% Conf. int.	$1.40 \times 10^5 - 3.4 \times 10^5$	$8.35 \times 10^2 - 3.18 \times 10^4$
Range	$4.90 \times 10^0 - 4.9 \times 10^7$	$1.34 \times 10^0 - 7.08 \times 10^6$
No. of samples	111	82
Coef. of var.	0.19	0.42
Dissolved oxygen, mg/l		
Mean	-	6.7
95% Conf. int.	-	6.0-7.4
Range	-	4.0-9.5
No. of samples	0	20
Coef. of var.	-	0.24
pH		
Median	7.2	7.5
Range	6.6-8.0	6.5-8.3
No. of samples	112	83

* Average hydraulic loading rate: 150 lpd/m^2.

The potential eutrophication of the receiving body of water should be considered when evaluating the performance of a sand filter. The reduction of macronutrients, including inorganic nitrogen and phosphorus, would be a beneficial function of any surface discharging wastewater treatment system. The reduction of ammonia would also be beneficial owing to the toxic effect of ammonia on fish.

Although there was some reduction in total nitrogen, the sand filter failed to remove a large percentage of the nitrogen present in the septic tank effluent (Table F-3). Total nitrogen in the septic tank effluent averaged 41.5 mg/l, with 87% in the form of ammonia. The sand filter effluent had an average total nitrogen concentration of 37.5 mg/l, a 9.6% reduction through the filter. Nitrification was virtually complete, however, resulting in an average nitrate + nitrite nitrogen concentration of 35.3 mg/l, or 94% of the total nitrogen present in the sand filter effluent. Because nitrate levels exceeded the safe drinking water standard of 10 mg-N/l (USEPA, 1974), it is recommended that disinfected sand filter effluent should not be discharged near wells or into creviced bedrock, even though the effluent may be biologically safe.

Total phosphorus was removed an average of 28% through the sand filter resulting in an effluent concentration of 14.1 mg-P/l. This performance is consistent with other sand filters studied where total phosphorus removal ranged from 20 to 30% (Sauer, 1975).

Aerobic Unit/Sand Filter System

Because two different hydraulic loading rates were used during the evaluation of the performance of the aerobic unit/sand filter system, the analysis of the data was separated into two distinct phases. Phase I, from April to September 1982, represents the time period when leaking plumbing resulted in an excessive volume of wastewater, and the sand filter was loaded at an average rate of 305 lpd/m^2 (7.5 gpd/ft^2). The performance of the aerobic unit cannot be evaluated because of this dilution effect; however, the data can be analyzed to evaluate the performance of the sand filter under various organic and hydraulic loading rates. Phase II represents the time period of October 1982, to September 1983. The leaking plumbing had been repaired and the sand filter was loaded at an average rate of 49 lpd/m^2 (1.2 gpd/ft^2).

The BOD_5 concentration of the aerobic unit effluent was relatively low during both phases of the study (Table F-4), when compared with other reported values (Otis & Boyle, 1976; Effert, 1977). During Phase I, the aerobic unit effluent averaged 25 mg/l, and while this low value might be explained by a dilution effect caused by the leaking plumbing, this does not explain the low aerobic effluent values reported in Phase II, which averaged 28 mg/l. During both phases, the sand filter produced a high quality effluent with respect to BOD_5 concentrations, and like the septic tank/sand filter samples, 100% of these effluent samples also achieved the Ohio Administrative Code value of 20 mg/l.

TABLE F-3. NITROGEN AND PHOSPHORUS CONCENTRATION, SEPTIC TANK/ SAND FILTER SYSTEM

Parameter	Septic tank effluent Apr 1982-Sep 1983	Sand filter effluent Apr 1982-Sep 1983*
NH_3-N, mg/l		
Mean	36.3	0.5
95% Conf. int.	34.3-38.3	0.4-0.6
Range	2.2-68.4	<0.1-3.2
No. of samples	115	80
Coef. of var.	0.30	0.94
TKN, mg/l		
Mean	40.7	3.3
95% Conf. int.	38.5-42.9	1.5-5.1
Range	2.9-67.7	<0.1-49.1
No. of samples	114	83
Coef. of var.	0.29	2.56
NO_2 + NO_3-N, mg/l		
Mean	0.6	35.3
95% Conf. int.	0.5-0.7	31.0-40.4
Range	<0.1-3.1	<0.1-110
No. of samples	115	85
Coef. of var.	1.08	0.61
Total-N, mg/l		
Mean	41.5	37.5
95% Conf. int.	39.4-43.6	32.9-42.1
Range	6.9-67.8	3.2-113
No. of samples	111	82
Coef. of var.	0.27	0.57
Total-P, mg/l		
Mean	19.5	14.1
95% Conf. int.	17.9-20.9	12.6-15.6
Range	1.2-39.7	1.8-32.6
No. of samples	116	82
Coef. of var.	0.42	0.49

* Average hydraulic loading rate: 150 lpd/m^2.

There was little variability in the sand filter effluent with respect to COD concentration during either Phase I or Phase II of the study. At the 305 lpd/m^2 (7.5 gpd/ft^2) loading rate, the COD concentration averaged 45 mg/l, while at the 49 lpd/m^2 (1.2 gpd/ft^2) loading rate, the concentration averaged 43 mg/l. The range of values throughout the study, 12 to 96 mg/l, is small, as is the coefficient of variation of less than 0.40. The low COD concentration and absence of variability in the sand filter effluent are probably due to the low average COD concentration and lack of variability in the aerobic unit effluent. In any event, the aerobic unit/sand filter system produced an effluent that had a consistently low COD.

TABLE F-4. EFFLUENT QUALITY DATA, AEROBIC UNIT/SAND FILTER SYSTEM

Parameter	Aerobic unit effluent Apr 1982-Sep 1982	Sand filter effluent Apr 1982-Sep 1982*	Aerobic unit effluent Oct 1982-Sep 1983	Sand filter effluent Oct 1982-Sep 1983†
BOD_5, mg/l				
Mean	25	4	28	2
95% Conf. int.	17-33	1-6	20-36	2-2
Range	2-112	1-6	2-94	1-7
No. of samples	26	13	44	53
Coef. of var.	0.88	1.7	0.91	0.50
COD, mg/l				
Mean	86	45	63	43
95% Conf. int.	72-100	39-51	58-68	38-47
Range	20-137	19-69	4-161	12-96
No. of samples	22	21	70	57
Coef. of var.	0.39	0.33	0.36	0.38
Suspended solids, mg/l				
Mean	32	61	38	24
95% Conf. int.	23-40	44-78	33-43	18-30
Range	10-77	7-169	3-93	2-92
No. of samples	22	23	62	62
Coef. of var.	0.63	0.68	0.50	1.0
Fecal coliforms, No. per 100 ml				
Geometric mean	1.25×10^4	4.9×10^2	5.89×10^3	1.23×10^2
95% Conf. int.	$4.24 \times 10^3 - 3.67 \times 10^4$	$2.16 \times 10^2 - 1.11 \times 10^3$	$3.26 \times 10^3 - 1.06 \times 10^4$	$6.57 \times 10^1 - 2.30 \times 10^2$
Range	$1 \times 10^0 - 7.94 \times 10^5$	$2 \times 10^0 - 2.29 \times 10^4$	$2.29 \times 10^0 - 5.37 \times 10^5$	$1 \times 10^0 - 9.12 \times 10^4$
No. of samples	25	25	72	62
Coef. of var.	0.29	0.34	0.29	0.52
Dissolved oxygen, mg/l				
Mean	-	-	3.4	7.1
95% Conf. int.	-	-	2.6-4.2	6.7-7.9
Range	-	-	0.6-8.5	5.0-8.2
No. of samples	0	0	43	19
Coef. of var.	-	-	0.76	0.18
pH				
Median	7.3	7.7	7.4	7.7
Range	6.9-7.8	7.0-8.1	6.7-8.1	6.8-8.2
No. of samples	25	26	73	64

* Average hydraulic loading rate: 305 lpd/m^2
† Average hydraulic loading rate: 49 lpd/m^2

Whereas the variation in the hydraulic loading rate had no observable effect on the BOD_5 and COD concentrations, it appears to have affected the suspended solids concentrations. During Phase I, April through September 1982, when the hydraulic loading rate averaged 305 lpd/m^2 (7.5 gpd/ft^2), there was an unexpected 91% increase in the suspended solids concentration through the sand filter, resulting in an average concentration of 61 mg/l in the sand filter effluent. A similar increase in suspended solids did not occur through the sand filter treating the septic tank effluent at a loading rate of 150 lpd/m^2 (3.7 gpd/ft^2), even though the sand used in both filters came from the same truck load. Further, when the hydraulic loading rate was reduced from 305 lpd/m^2 (7.5 gpd/ft^2) to 49 lpd/m^2 (1.2 gpd/ft^2), the suspended solids concentration in the sand filter effluent decreased substantially. It seems reasonable to conclude that, in this study, the hydraulic loading rate was an important factor in the suspended solids concentration of the sand filter effluent. It is possible that fines may have been washed from the sand when the filter was receiving pretreated wastewater at the higher loading rate.

Fecal coliform reduction through the sand filter averaged 96 and 98% respectively, during Phase I and Phase II of the study. Although a high percentage of the fecal coliforms was removed by the sand filter during Phase I, the fecal coliform arithmetic log mean value of the sand filter effluent still exceeded surface water standards for contact recreation use. During Phase II, the sand filter effluent did achieve the contact water standard. However, disinfection of the effluent before it is discharged into a watercourse is recommended because even though the arithmetic log mean value of the effluent was less than 200 fecal coliforms per 100 ml, 31% of the samples tested exceeded this limit.

Dissolved oxygen concentrations were not measured during the April through September 1982, period, but measurements were made during Phase II of the study, from October 1982, to September 1983. During that time, the aerobic unit produced an effluent that had an average D.O. of 3.4 mg/l; only 35% of the aerobic unit effluent samples achieved a D.O. of 4.0 mg/l or greater. The dissolved oxygen concentration of the sand filter effluent averaged 7.1 mg/l, with 100% of the samples achieving a D.O. of 4.0 mg/l or greater.

The median pH of the aerobic unit/sand filter effluent was 7.3 and 7.7, respectively. One hundred percent of the samples tested, from either location, had a pH within the safe range for fish protection.

At the 49 lpd/m^2 (1.2 gpd/ft^2) loading rate, there was a 9.6% reduction in total nitrogen through the sand filter, as determined by the mean concentrations, resulting in an effluent total nitrogen concentration of 16.7 mg/l (Table F-5). Nitrification was virtually complete, with 95% of the total nitrogen in the nitrate or nitrite form. At the 305 lpd/m^2 (7.5 gpd/ft^2) loading rate, the data show that on average, the concentration of total nitrogen in the sand filter effluent was 10% greater than the aerobic unit effluent. Composite samples had been collected during the filter runs, and the automatic sampling mechanism was activated only during a dosing event. It therefore is believed that the samples collected are representative, and that the data are accurate. It is speculated that, at the higher loading rates, nitrogenous material present in the sand may have been washed out of the filter, thereby increasing the total nitrogen concentration in the filter effluent. Even at this high loading rate, nitrification was 90% complete, indicating that any material that may have been washed from the filter was in the oxidized state.

Throughout the study, the total phosphorus concentration in the aerobic unit effluent was extremely low, as compared with other values reported in the literature (Otis & Boyle, 1976; Effert, 1977). During Phase I, these low values might be attributed to the dilution of the wastewater because of the leaking plumbing. During Phase II, the concentration was still low, probably because the residents did not do their laundry at home. In any event, because the total phosphorus concentration was extremely low throughout the study, and not representative of typical aerobic unit effluent, the 48 to 68%

reduction in total phosphorus through the sand filter is probably not representative of the performance of a typical intermittent sand filter treating aerobic unit effluent.

TABLE F-5. NITROGEN AND PHOSPHORUS CONCENTRATION, AEROBIC UNIT/SAND FILTER SYSTEM

Parameter	Aerobic unit effluent Apr 1982-Sep 1982	Sand filter effluent Apr 1982-Sep 1982*	Aerobic unit effluent Oct 1982-Sep 1983	Sand filter effluent Oct 1982-Sep 1983†
NH_3-N, mg/l				
Mean	3.9	0.4	8.4	0.2
95% Conf. int.	2.2-5.6	0.1-0.7	6.7-10.0	0.2-0.3
Range	<0.1-17.5	<0.1-4.7	<0.1-27.4	<0.1-0.6
No. of samples	29	28	78	65
Coef. of var.	1.22	2.02	0.89	0.67
TKN, mg/l				
Mean	6.4	1.4	9.8	0.8
95% Conf. int.	4.4-8.4	1.0-1.8	8.1-11.5	0.6-1.0
Range	0.3-19.9	0.2-4.8	<0.1-30.6	<0.1-5.6
No. of samples	29	28	78	63
Coef. of var.	0.84	0.78	0.80	1.13
NO_2 + NO_3-N, mg/l				
Mean	4.2	10.4	7.4	15.8
95% Conf. int.	3.0-5.4	8.5-12.3	6.6-8.2	14.3-17.3
Range	<0.1-12.0	<2.7-29.0	<0.1-18.0	4.4-32.5
No. of samples	29	29	77	64
Coef. of var.	0.75	0.49	0.50	0.40
Total-N, mg/l,				
Mean	10.5	11.6	17.8	16.7
95% Conf. int.	9.1-11.9	9.5-13.7	15.7-19.9	15.2-18.2
Range	4.1-20.0	4.3-30.2	8.2-35.1	8.5-32.7
No. of samples	29	29	75	61
Coef. of var.	0.35	0.49	0.51	0.38
Total-P, mg/l				
Mean	1.9	0.6	5.3	2.8
95% Conf. int.	1.6-2.2	0.5-0.7	4.5-6.1	2.1-3.5
Range	0.6-4.3	<0.1-2.1	<0.1-15.9	<0.1-11.2
No. of samples	29	29	78	66
Coef. of var.	0.44	0.66	0.71	0.97

* Average hydraulic loading rate: 305 lpd/m^2.

† Average hydraulic loading rate: 49 lpd/m^2.

Sand Filter Maintenance

Neither sand filter required extensive servicing during the 22-month period of the study. Some work was done to the filters, however, which can be classified into two categories: maintenance to the filter basin for aesthetic purposes, and maintenance to the sand media for extended filter life.

Maintenance to the concrete filter basins consisted of the removal and the replacement of the 18-mm (0.75-in) plywood covers, which had become warped and discolored within a few months. During the summer of 1983, the plywood covers were replaced with 51 mm x 152 mm (2 in x 6 in) treated deck lumber, which is less subject to warping, and should last a number of years before it needs to be replaced. The deck lumber is also more aesthetically attractive and structurally sound.

Routine or scheduled maintenance of the sand media was not necessary during the 22-month study. This was unexpected because intermittent sand filters having similar media characteristics and organic and hydraulic loading rates have been reported to require some type of servicing every 6 months (USEPA, 1980). Typical servicing would include raking or removing the first few inches of sand, or taking the filter out of service for an extended period of time to allow the organic matter in the sand to decompose. Both procedures would remove the crusting layer that develops on the surface of the sand and restore the sand's infiltrative capacity. Maintenance is typically required when the pretreated wastewater stays ponded on the surface of the sand between dosing events.

Although neither sand filter required periodic maintenance, the filter treating the aerobic unit effluent was serviced during the study. In December 1982, 10 months after the filter was first used, it was observed that the aerobic unit effluent was not being distributed evenly over the surface of the filter, and that the unequal distribution of the effluent was being caused by the configuration of the concrete filter basin. The filter basin had a 10 cm (4 in) thick baffle that extended from above the sand to a depth of 50.8 cm (20 in) into the sand, thus separating the basin into two equally sized areas. An equal volume of effluent was not being applied to each half of the filter during dosing, and because of the baffle, the liquid could not flow to the half of the filter that was receiving less effluent. The unequal application of aerobic unit effluent onto the filter caused the formation of a 13 mm (0.5 in) thick crust on the half of the filter receiving the greatest volume of effluent. The crust caused effluent to pond on this side of the filter to a depth of 51 to 67 mm (3 to 4 in) following a dosing event, while the other half of the filter showed no signs of a crust, and effluent did not pond.

When this phenomenon was observed, a valve was installed in the distribution pipe, resulting in an equal distribution of the effluent over both halves of the filter. At the same time, the sand was raked to a depth of 10 cm (4 in), and from December 1982 through September 1983, both halves of the sand filter received an equal volume of aerobic unit effluent and a crust never redeveloped.

A concrete baffle was not present in the sand filter basin treating the septic tank effluent, and as a result, effluent was applied evenly over the entire surface area of the filter. A crusting layer never developed on this filter and the sand media never required raking. Because of the difficulty in distributing liquid evenly over a sand filter, a baffle is not recommended if structural support of the filter basin is required.

Construction and O&M Costs

The performance of an onsite wastewater treatment system is an important factor in the recommendation and approval of the technology by the local regulatory agencies. Equally important to the homeowner is the cost of the system, both the initial cost and the annual operation and maintenance costs. These two costs, amortized together at 10% per year over the expected life of the system, will give the total annual costs.

Table F-6 shows the total annual costs of each wastewater treatment system. These costs do not include labor for the initial installation of the systems studied, because labor costs including the hook-up to the home, can be very site specific and may skew the overall costs. Because each system has three major components, the aerobic unit or septic tank, the pumping chamber, and the sand filter, given similar construction sites, excavation and other installation labor costs would be nearly the same, regardless of the treatment system.

TABLE F-6. ANNUAL MATERIAL, OPERATION AND MAINTENANCE COSTS

	System costs, 1982 dollars*	
Item	Aerobic unit and sand filter	Septic tank and sand filter
Septic tank	-	$ 72†
Aerobic unit	$ 359§	-
Pumping chamber/pump	158**	158**
Sand filter, 6.3 m^2 (68 ft^2)	-	153††
Sand filter, 3.2 m^2 (34 ft^2)	80††	-
Total annual costs	$ 597	$ 383

* Material costs, amortized at 10% over 20 yr, plus O&M costs. The cost of labor for the installation of the systems is not included.
† O&M costs include tank pumping once every 3 yr.
§ O&M costs include annual tank pumping, 550 kW-hr of electricity each year @ $0.09 per kW, and aerator replacement every 10 yr.
** Assumes replacement of the effluent pump every 10 yr.
†† Assumes 5 hr of labor for maintenance/year, @ $6.00/hr.

Previous research has found that an intermittent sand filter treating the effluent from a home aerobic unit can be hydraulically loaded at twice the rate of a similar filter treating septic tank effluent without increasing annual maintenance costs (USEPA, 1980). As a result, the cost of the aerobic unit/sand filter system listed in Table F-6 is based on a filter one-half the size of a sand filter treating an equal volume of septic tank effluent. Even with a reduction in filter size, over its

20-year life, the aerobic unit/sand filter system would cost $214 more to operate per year than the septic tank/sand filter system. The higher annual cost to operate this system can be directly attributed to the capital and O&M costs of the aerobic unit.

SUMMARY AND CONCLUSIONS

Both of the intermittent sand filters produced a high quality effluent with respect to BOD_5, suspended solids, and COD, regardless of the type of pretreatment. The effluent from the filters usually met the State of Ohio discharge standards for BOD_5 and suspended solids. Neither the septic tank nor the aerobic unit were able to achieve this standard without the sand filters.

More than 95% of the fecal coliforms were removed as the pretreated wastewater passed through the sand filters. However, fecal coliform numbers in the septic tank/sand filter effluent were still high and disinfection is recommended.

Neither filter removed a large percentage of total nitrogen from the effluents, but nitrification was virtually complete. A more significant reduction was seen in the removal of phosphorus by the filters.

At the loading rates studied, neither sand filter required periodic maintenance over the 22-month period of this study. Dosing the effluent onto the filters and allowing the sand to drain between dosing events was probably responsible for these long filter runs.

Capital and operation and maintenance costs for the aerobic unit/sand filter system are significantly greater than the cost of the septic tank/sand filter system. The higher costs, amounting to an additional $214 per year over a 20-yr life, can be directly attributed to the capital costs of the aerobic unit, and to the substantially higher O&M costs resulting from energy consumption and aerator replacement.

Because of the significantly higher capital and O&M costs of the aerobic unit/sand filter system, with no corresponding increase in the level of wastewater treatment, a septic tank appears to provide the most acceptable level of treatment of the two pretreatment systems studied.

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Appendix G

EFFECTS OF VARIOUS HYDRAULIC LOADING RATES ON THE EFFLUENT QUALITY FROM INTERMITTENT SAND FILTER COLUMNS*

David Janik

INTRODUCTION

Many areas in the United States are unsuitable for the installation and use of soil absorption fields owing to the presence of physical site limitations such as impermeable soils, a high water table, or inadequate lot size. If onsite wastewater disposal is to be used at these locations, it is necessary to consider alternatives to conventional practices. One such alternative, involving the treatment and surface discharge of the wastewater, is the intermittent sand filter.

When designed and operated according to accepted standards, intermittent sand filters produce a high quality effluent. There is a considerable amount of flexibility within these design standards, however, and if it is desirable to produce the highest quality effluent possible, design criteria should be better defined.

The objective of this study was to evaluate the effect of varying the hydraulic loading rate on the performance of sand columns that were used to simulate full-scale intermittent sand filters. The loading rates selected ranged from 20 to 162 lpd/sq m, and were based on the range of loading rates commonly recommended for intermittent sand filters (USEPA, 1980).

MATERIALS AND METHODS

Ten sand columns were used in this study to simulate the performance of five full-scale intermittent sand filters. The columns, installed in duplicate, were 1.5 m long and constructed of 10.2-cm diameter PVC drain and sewer pipe. This provided a surface area of 81 sq cm.

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* Extended abstract from thesis. Department of Civil and Environmental Engineering. University of Cincinnati. 1985.

The columns were prepared for this study in the following manner. Each column was capped on the bottom, and gas sampling holes were drilled into the sides to facilitate the collection of gas samples for gas chromatograph analysis (Figure G-1). Septums were glued over the gas sampling holes and their presence permitted the use of an airtight glass syringe to collect gas samples from within the columns.

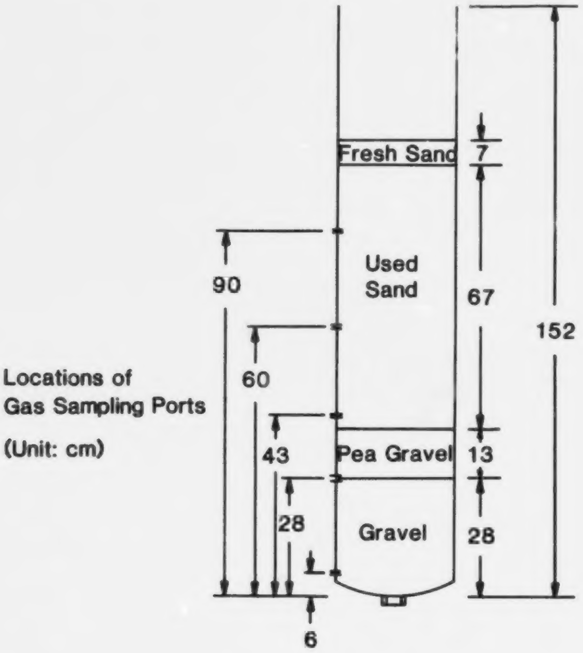


Figure G-1. Cross section of a sand-filled column showing gas sampling holes.

The interiors of the columns were washed and then loaded with the fill material. Each column was first loaded with a mixture of prewashed gravel and crushed stone to a depth of 28 cm. Above this material, a 13-cm layer of prewashed pea gravel was laid, and above the pea gravel there was 74 cm of sand, which had an effective size of 0.44 mm and a uniformity coefficient of 2.5. The lower 66 cm of the sand had been taken from an operating intermittent sand filter that was showing greater than 95% nitrification and less than 5% denitrification (Effert and Cashell, 1984), while fresh sand was used in the top 8 cm of the column. The used sand was taken from an operating sand filter in order to minimize the typical 30- to 45-day start-up period required to establish a population of nitrifying bacteria. The new sand was used on top of the columns to reduce the chances of filter clogging at the surface.

When fully assembled, the columns were put into service. This required the collection of septic tank effluent, which was obtained once per week from a residence and placed in polypropylene tanks where it was held at room temperature until it was applied onto the filters through a series of pumps and timers based on the hydraulic loading rates listed in Table G-1. At the end of the week, excess effluent not applied to the sand columns was discarded, and the holding tanks were filled with fresh effluent.

TABLE G-1. COLUMN LOADING RATES AND APPLICATION VOLUMES

Columns	Hydraulic loading rate, lpd/sq m	Application vol. per cycle, ml	Application vol. per day, ml
1A & 1B	20	55	165
2A & 2B	51	138	413
3A & 3B	81	220	661
4A & 4B	122	330	990
5A & 5B	162	440	1320

Influent and effluent sample collection began immediately after the columns were put into service, but except for COD, the data presented in this paper reflect the performance of the sand columns after they had been in operation for 5 weeks. Additional data concerning the performance of the sand columns during the 5-week start-up period is presented for COD.

Sample collection occurred during a dosing event for the column influent or feed, and following a dosing event after the wastewater had passed through the sand for the column effluent. All samples were collected in chilled cubitainers that contained concentrated sulfuric acid in a ratio of 1 ml acid to 1 liter of sample. Samples were obtained two or three times a week for a period of 6 months and routinely analyzed for chemical oxygen demand (COD), ammonia (NH_3-N), nitrate (NO_3-N), nitrite (NO_2-N), total Kjeldahl nitrogen (TKN), and total phosphorus (Total-P) by the U.S. Environmental Protection Agency, Cincinnati, Ohio. Occasionally, unpreserved grab samples were also collected in sterile containers and tested for fecal coliforms and total suspended solids by the staff of the Department of Civil and Environmental Engineering at the University of Cincinnati.

RESULTS AND DISCUSSION

The results of this study are presented separately for each of the parameters evaluated. A summary showing average concentrations of septic tank effluent and sand filter column effluent is presented in Table G-2. A summary of column efficiency, defined as the percent removal of the parameter, is presented in Table G-3.

TABLE G-2. MEAN SAND FILTER COLUMNS INFLUENT AND EFFLUENT CONCENTRATIONS, MG/L

Parameter	Influent	20 lpd/sq m 1A	1B	51 lpd/sq m 2A	2B	81 lpd/sq m 3A	3B	122 lpd/sq m 4A	4B	162 lpd/sq m 5A	5B
TSS	24	2	3	3	3	3	2	3	3	2	1
Fecal coliforms*	4.8	0	0	0	0	0	0	0.3	0.8	2.7	3.0
Total-P	11.4	4.0	3.5	6.6	6.7	7.6	7.7	8.5	8.6	9.0	8.9
COD	226	39	29	28	27	30	28	30	28	30	30
NH_3-N	66.6	0.6	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.2	0.1
TKN	74.2	3.3	1.6	1.4	1.4	1.4	1.4	1.3	1.3	1.5	1.5
NO_3-N	0.2	95.5	96.8	86.0	86.3	83.9	81.9	82.0	81.4	82.0	76.6
Total-N	74.5	98.4	98.4	87.5	86.7	85.0	83.3	83.6	82.8	83.5	81.3

* Log, per 100 ml.

TABLE G-3. SAND FILTER COLUMN EFFICIENCIES, PERCENT REMOVAL OF COMMON WASTEWATER CONSTITUENTS

Parameter	20 lpd/sq m 1A	1B	51 lpd/sq m 2A	2B	81 lpd/sq m 3A	3B	122 lpd/sq m 4A	4B	162 lpd/sq m 5A	5B
TSS	92	88	88	88	88	92	88	88	92	96
Fecal coliforms*	99+	99+	99+	99+	99+	99+	94	83	44	38
Total-P	65	69	42	41	33	32	25	25	21	22
COD	83	88	88	88	87	88	87	88	87	87
TKN	96	98	98	98	98	98	98	98	98	98
Total-N†	32	32	17	16	14	12	12	11	12	9

* Log, per 100 ml.
† An increase occurred for this parameter.

Total Suspended Solids

The septic tank effluent applied to the sand filter columns had an average total suspended solids concentration of 24 mg/l, which is about half the normal concentration for septic tank effluent (USEPA, 1978). It is hypothesized that the suspended solids concentration was low because the liquid sat unagitated in a tank prior to its application onto the sand columns. This lack of disturbance of the liquid probably allowed for a greater amount of settling than would occur within a septic tank.

All of the columns produced an effluent with a very low total suspended solids concentration, with average values ranging from 1 to 3 mg/l. Within the range of 20 to 162 lpd/sq m, the hydraulic loading rate had no effect on the suspended solids concentration of the column effluent.

Fecal Coliforms

The hydraulic loading rate had an effect on fecal coliform removal by the sand-filled columns. At loading rates of 81 lpd/sq m or less, 100% of the filter effluent samples tested had less than the detectable limit of 2 fecal coliforms per 100 ml. At the two higher loading rates, coliforms were found in the column effluent, and their numbers increased with an increase in the loading rate. At the 122 lpd/sq m loading rate, the columns produced an effluent that had an average of less than 7 fecal coliforms per 100 ml, which is far less than the standard of 200 fecal coliforms per 100 ml for recreational water contact (Federal Water Pollution Control Administration, 1968). The sand column receiving septic tank effluent at the highest loading rate of 162 lpd/sq m failed to produce an effluent that had an average of less than 200 fecal coliforms per 100 ml. These data suggest that sand filters should be designed with low hydraulic loading rates to reduce the probability of fecal coliforms passing through the filter and appearing in the effluent.

Total Phosphorus

The septic tank effluent had an average phosphorus concentration of 11.4 mg/l, which is within the range of values reported in the literature (USEPA, 1978).

There was a direct relationship between phosphorus removal and the hydraulic loading rate; the columns with the lower loading rates were more efficient in removing phosphorus than the columns with the higher loading rate (Figure G-2). An analysis of the data shows that at the three lowest loading rates of 20, 51, and 81 lpd/sq m, the Total-P concentrations of the column effluent were significantly different from each other, as well as from the two higher loading rates. However, at the higher loading rates of 122 and 162 lpd/sq m, the Total-P concentrations were significantly different from the lower loading rates, but they were not different from each other. Total phosphorus removal did not improve when the hydraulic loading rate was reduced from 162 to 122 lpd/sq m, but improvement in treatment did occur with each subsequent lowering of the hydraulic loading rate from 81 to 20 lpd/sq m. There was no significant difference in columns with the same loading rates.

In this study, the lower hydraulic loading rates produced effluent with a lower concentration of total phosphorus. In areas where phosphorus discharge into the environment is of concern, the hydraulic loading rate should be considered when design specifications for intermittent sand filters are being developed.

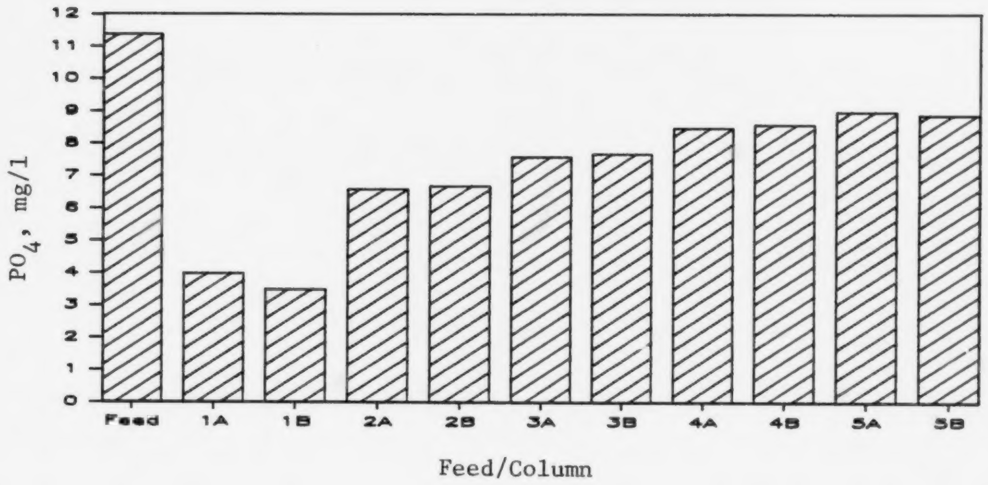


Figure G-2. Average influent and effluent total phosphorus concentrations.

Chemical Oxygen Demand

During the initial 5 weeks of this study, septic tank effluent with an extremely high COD concentration was inadvertently applied to the sand columns. The average COD concentration was 1161 mg/l, which is more than 12 times the COD concentration of typical septic tank effluent (USEPA, 1978). The data obtained during this period were not used to determine the treatment efficiencies of the sand columns with respect to the other parameters. These data are discussed with respect to COD, however, because the columns were able to successfully treat the shock loads, thereby reducing the COD by more than 90%, as shown in Table G-4. The periodic shock load of a short duration did not have a detrimental effect on the column's operation or performance in this study.

TABLE G-4. COLUMN INFLUENT AND EFFLUENT COD QUALITY DURING A PERIOD OF SHOCK LOADING

Measurement	Influent	20 lpd/sq m 1A	1B	51 lpd/sq m 2A	2B	81 lpd/sq m 3A	3B	122 lpd/sq m 4A	4B	162 lpd/sq m 5A	5B
Mean, mg/l	1161	58	60	70	64	59	61	94	80	109	86
% reduction	----	95	95	94	95	95	95	92	93	91	93

The results of the COD data obtained following the period of shock loading are presented in Table G-4. COD removal by the sand columns averaged between 83 and 88% (Table G-3), and performance appeared to be consistent among columns regardless of the hydraulic loading rate.

The results indicate that between 20 and 162 lpd/sq m, the hydraulic loading rate had no effect on the COD removal efficiency of the sand columns.

Ammonia Nitrogen

All of the sand columns produced a consistently high quality effluent with respect to ammonia. Even though the septic tank effluent averaged 66.6 mg/l, which is significantly higher than what most other researchers have reported (USEPA, 1978), 9 of the 10 columns produced effluent that averaged less than 0.2 mg/l. Only one of the columns averaged 0.6 mg/l. All of the columns reduced ammonia by 99% or more, indicating that nitrification was almost complete. These results clearly indicate that variations in the hydraulic loading rate, at least within the range evaluated in this study, had no effect on column effluent quality for ammonia.

Total Kjeldahl Nitrogen

The treatment efficiency of the sand columns and their ability to remove total Kjeldahl nitrogen were similar to their ability to remove ammonia. The columns reduced TKN an average of 96 to 98% (Table G-3), which is approximately 2 percentage points lower than the ammonia reductions. This slight difference in ammonia vs. TKN removal was expected because organic nitrogen, measured in the TKN analysis, must first be converted to ammonia before it can be nitrified. This conversion process is slower than nitrification, and, as a result, the removal of TKN is slightly less efficient than the removal of ammonia. As with ammonia, the hydraulic loading rate had no effect on column effluent quality.

Nitrate Nitrogen

Owing to anaerobic conditions, the septic tank effluent had a very low average nitrate-N concentration of 0.2 mg/l with no single sample having a concentration greater than 1.0 mg/l. As a result of nitrification within the sand, the column effluent showed a large increase in nitrate-N, with average values ranging from 79.6 to 96.8 mg/l.

An analysis of the data shows that there was no statistically significant difference between columns with the same loading rates. However, the columns receiving septic tank effluent at the 20 lpd/sq m loading had a statistically significant higher nitrate concentration than the other four loading rates. Although the means are different, there was no statistically significant difference in nitrification between columns loaded at 51, 81, 122, and 162 lpd/sq m.

Total Nitrogen

Total nitrogen was not analytically determined in this study, rather it was calculated by summing the various nitrogen parameters for any given sample in an attempt to conduct a nitrogen balance for the columns. All 10 columns in this study had a net nitrogen gain that ranged from 9 to 32% (Table G-3).

An analysis of the data indicates that there is no significant difference between the columns with the same loading rate. There is also no difference between the 51, 81, 122, and 162 lpd/sq m loading rates with respect to total nitrogen. However, the columns receiving effluent at the 20 lpd/sq m loading rate had significantly more total nitrogen than the others.

The reasons for this net increase in nitrogen are unknown, but two possible explanations are offered. First, an error in the analysis of nitrate is possible. Measurements of the accuracy and precision of the analytical tests, conducted throughout the study (Janik, 1985), showed that approximately 10% of the Total-N gain could be explained by poor accuracy. However, it is highly unlikely that analytical error alone could account for the net nitrogen gain. Second, it has been speculated that nitrogen fixation occurred within the columns. Further testing was not done, and the theory of nitrogen fixation, has not been proven.

Column Gases

Gas samples were collected from various levels of the columns and analyzed on a Perkin-Elmer 900 Gas Chromatograph with a thermal conductivity detector and a two-phase column for percent carbon dioxide, oxygen, and nitrogen. The purpose of these analyses was to locate any anaerobic areas within the column profile; to see if any areas of high nitrogen gas existed; and to obtain an overall understanding of the gaseous environment within the sand column.

The general findings of these analyses were that the percent nitrogen gas within the columns ranged from 77 to 80, which is approximately the same as ambient air. The percent nitrogen in the columns did not appear to be affected by either the hydraulic loading rate or by the level in the column from which the sample was taken.

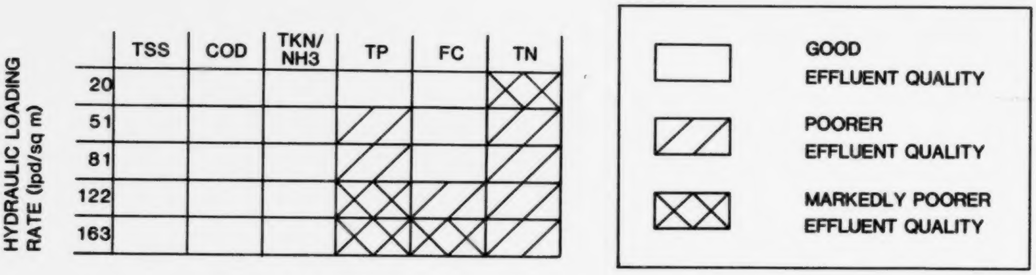
The percent of oxygen within the column was more variable than nitrogen, and two major trends were observed. First, samples taken from columns being fed the higher loading rates had a lower percentage of their column gas as oxygen. Second, oxygen levels decreased the lower into the column the sample was taken. These trends appear to be the logical results of increased biological activity when more waste is applied, and greater difficulty of sand aeration at lower depths in the filter, respectively. Anaerobic conditions were never observed, and oxygen ranged from 12 to 21% of the gas in the column.

Carbon dioxide, like oxygen, also varied and ranged from 1 to 8% of the total column gas. Trends opposite that of oxygen were seen, however. The greater the volume of wastewater applied to the column, the greater the percent of carbon dioxide that was observed in the column gas. Also, the lower into the column the sample was taken, the greater was the percent of carbon dioxide.

SUMMARY AND CONCLUSIONS

Five pairs of sand columns were used to simulate intermittent sand filters for the purpose of determining the effect of hydraulic loading on effluent quality. In this study, the sand was calcareous and probably originated from limestone parent material, and it had an effective size of 0.44 mm with a uniformity coefficient of 2.5. The hydraulic loading rates selected were 20, 51, 81, 122, and 162 lpd/sq m.

The results of the study are summarized in Figure G-3. The hydraulic loading rate was not a factor in the quality of the effluent produced by the sand columns for four of the parameters tested: total suspended solids, chemical oxygen demand, total Kjeldahl nitrogen, and ammonia. Three other parameters tested appeared to be affected, however. Total phosphorus concentration and fecal coliform numbers were significantly higher at the higher loading rates. Just the opposite occurred for total nitrogen, however. There appeared to be a net gain of nitrogen in the effluent coming from the columns, and the column with the lowest hydraulic loading rate showed the greatest gain in total nitrogen. As a result, for total nitrogen, effluent quality was significantly poorer than the others at the 20 lpd/sq m loading rate.



HYDRAULIC LOADING RATE (lpd/sq m)	TSS	COD	TKN/ NH3	TP	FC	TN
20	☐	☐	☐	☐	☐	▩
51	☐	☐	☐	▨	☐	▨
81	☐	☐	☐	▨	☐	▨
122	☐	☐	☐	▩	▨	▨
163	☐	☐	☐	▩	▩	▨

☐	GOOD EFFLUENT QUALITY
▨	POORER EFFLUENT QUALITY
▩	MARKEDLY POORER EFFLUENT QUALITY

Figure G-3. Summary of the effect of various hydraulic loading rates on the effluent quality from intermittent sand filter columns. (TP = Total-P, FC = fecal coliforms, TN = Total-N).

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APPENDIX H

THE EFFECT OF THE RECIRCULATION RATIO ON THE RECIRCULATING SAND FILTER*

Sekou Toure

INTRODUCTION

For many years, onsite wastewater treatment and disposal systems have provided sewage treatment in areas beyond the reach of centralized sewers. In many areas, however, soil absorption fields do not function properly because of high groundwater tables, impermeable soils, or shallow bedrock (Ralph et al., 1979). The resultant failure of these systems creates health hazards and nuisances.

In order to provide an alternative to onsite wastewater disposal systems in areas where they are not suitable, a modified intermittent sand filter, the recirculating sand filter (RSF) illustrated in Figure H-1, was developed by Favreau and Hines (Hines et al., 1977; Hines and Favreau, 1974; Ralph et al., 1979; Teske, 1978). The advantage of an RSF is that it provides satisfactory treatment without the odor potential of an intermittent sand filter.

Although the RSF has been shown to be a reasonable alternative, the most efficient recirculation ratio for the RSF has not been determined. The recirculation ratio of recirculated wastewater to raw wastewater must be in the range of 3 to 1 or greater to maintain aerobic conditions and to minimize odor, but it should be 5 to 1 or less to avoid keeping the filter too wet, causing plugging or ponding (Teske, 1980).

The objectives of this study were:

1. To determine the level of treatment of septic tank effluent achieved by a laboratory model of a recirculating sand filter;
2. To determine whether recirculation ratios of 3 to 1, 4 to 1, and 5 to 1, provided different levels of treatment.

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* Extended abstract from thesis. Department of Civil and Environmental Engineering. University of Cincinnati. 1985.

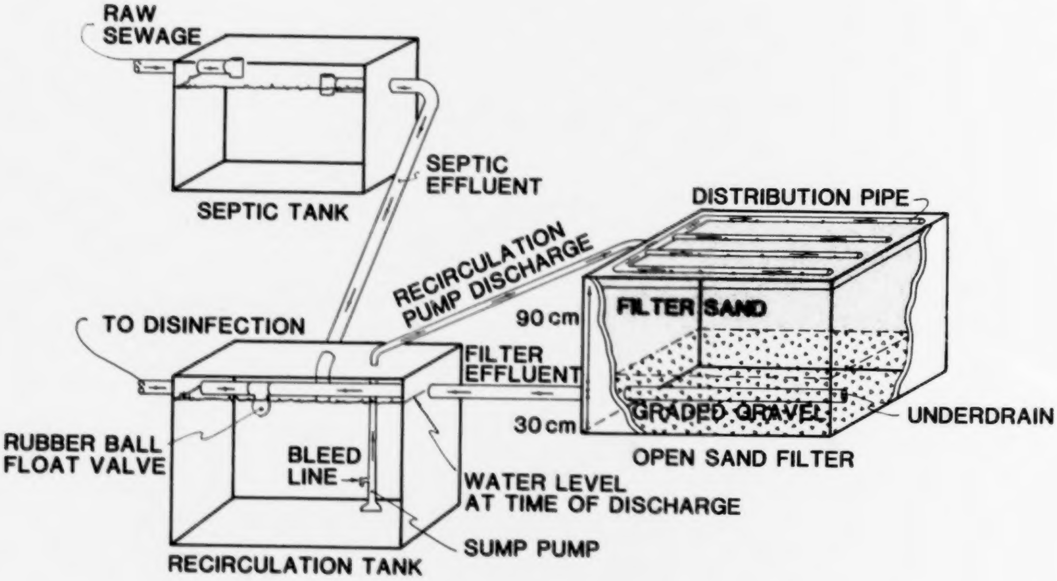
RAW SEWAGE
SEPTIC TANK
SEPTIC EFFLUENT
DISTRIBUTION PIPE
RECIRCULATION PUMP DISCHARGE
FILTER SAND
90 cm
TO DISINFECTION
FILTER EFFLUENT
GRADED GRAVEL
UNDERDRAIN
30 cm
OPEN SAND FILTER
RUBBER BALL FLOAT VALVE
BLEED LINE
WATER LEVEL AT TIME OF DISCHARGE
SUMP PUMP
RECIRCULATION TANK

Figure H-1. Schematic of a typical recirculating sand filter (from Hines and Favreau, 1975. Figure 1, p. 132).

MATERIALS AND METHODS

System Design

A laboratory model, shown in Figure H-2, was constructed to simulate the recirculating sand filter. Three systems consisting of a recirculation tank and a sand filter column were used. A common storage tank was used for all three systems. Septic tank effluent was collected weekly and stored in four 19-liter containers in the same room and at the same temperature as the model. An 18.9-liter dosing tank was used to distribute the septic tank effluent into the recirculation tanks. This tank was filled with storage tank effluent once a day using a small pump.

The recirculation tanks were constructed from Plexiglass and had a volume of approximately 5 liters. The tanks were designed to meet a minimum detention time requirement of one day's wastewater flow (Teske, 1978; Uebler, 1981; USEPA, 1980). Each tank was covered with a Plexiglass sheet into which a 7.6-cm diameter opening was drilled. A 7.6-cm diameter flood guard, which served as a float valve, was inserted into the opening. The flood guard was a circular rubber cylinder held by a moveable stem connected to a circular iron. The stem enabled the rubber cylinder to move vertically to open or close the hole on the cover of the tank, depending on the water level in the tank. In addition, a Plexiglass tube was sealed on the top of the flood guard. This tube had a 2.5-cm diameter opening on the side that was connected to the effluent discharge line. The effluent discharge line was also used as the sample collection port. The septic tank effluent was dosed into the recirculation tank through a 1.3-cm diameter hole drilled on the cover of the tank. Another 1.3-cm diameter hole 7.6 cm from the bottom on the side of the tank was connected to the recirculation pump. Tygon® tubing was used for all feed systems.

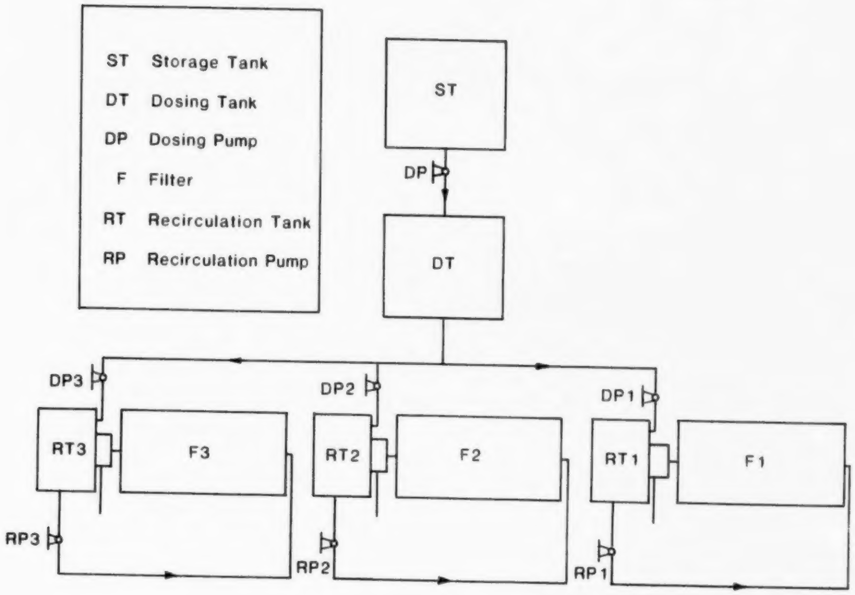

Figure H-2. Laboratory model of recirculating sand filter.

The sand filter columns were constructed of 15.2 cm-ID x 1.829 m x 0.6-cm thick Plexiglass. Each filter contained an underdrain located at the bottom of the column that collected the filter effluent to be returned to the recirculation tank.

The columns contained two distinct zones, the bedding material and the filter media. The bedding material consisted of 25.4 cm of washed graded gravel ranging in size from 1.3 to 2.5 cm. The gravel was in direct contact with the filter material, which consisted of 66 cm of sand with an effective size of 0.4 mm and a uniformity coefficient of 3.1.

The columns and the recirculation tanks were entirely covered with a dark paper to simulate field conditions as much as possible and to prevent physical changes within the columns and the tanks that might favor the growth of photosynthetic microorganisms.

Six metering pumps with capacities ranging from 0 to 11,200 ml/min and pressure ratings of 100 psig were used. Three of these pumps dosed the recirculation tanks with the septic tank effluent, and the other three were used as the recirculating pumps and pumped the wastewater from the recirculation tanks to the filters. All the pumps were connected to a programmable switch timer. The timer was programmed to activate the pumps once every hour.

A loading rate of 122 lpd/sq m based on the raw sewage flow was selected for this study. With the column surface area of 181 cm^2, this resulted in a design flow of 92 ml/hr.

Recirculation Ratio

The system with the recirculation ratio of 3 to 1 (system 1) had recirculation tank effluent dosed onto the filter once every hour for about 1 min at a rate of 276 ml/dose for a total of 6.6 l/day. Of the 276 ml of the recirculation tank effluent applied to the filter, 184 ml returned to the tank and 92 ml overflowed to the discharge line.

For the system with the recirculation ratio of 4 to 1 (system 2), recirculation tank effluent was dosed onto the filter once an hour for about 1 min at a rate of 368 ml/dose for a total of 8.8 l/day. Of the 368 ml applied to system 2, 276 ml returned to the tank and the remaining 92 ml discharged to the overflow line.

System 3, with a recirculation ratio of 5 to 1, was dosed once an hour for 1 min at a rate of 460 ml/dose for a total of 11.0 l/day. Of the 460 ml of the recirculation tank effluent applied to the filter, 368 ml were returned to the tank and 92 ml to the effluent discharge line.

PROCEDURES AND ANALYSES

The laboratory RSFs were monitored at room temperature for 12 weeks. Septic tank effluent was collected once a week from a family residence in the Cincinnati, Ohio, area. Collection was performed in a manner such that collecting sludge or scum was avoided.

Collection and Analysis of Samples

Septic tank effluent samples and column effluent samples were collected twice a week. All samples were analyzed for chemical oxygen demand (COD), total Kjeldahl nitrogen (TKN), total suspended solids (TSS), and pH, according to procedures listed in Standard Methods for the Examination of Water and Wastewater (American Public Health Association, 1980).

Except for samples for pH measurements, all samples were preserved with 1 ml/l of concentrated sulfuric acid added to the effluent sample containers before collection. The sample containers were also kept in a cooler during collection. Septic tank effluent was preserved by adding 1 ml/l of concentrated sulfuric acid and refrigerating samples prior to analysis.

RESULTS

Figures H-3, H-4, and H-5 show the sand filter influent (septic tank effluent) and the sand filter effluent COD, TKN, and TSS concentrations, respectively, for each sampling period. These results were then used to calculate average COD, TKN, and TSS concentrations and percentage removal obtained with the three RSFs. Table H-1 indicates these results, along with pH ranges.

COD concentrations were consistently reduced by all three RSFs throughout the study period (Figure H-3). The performance of the three RSFs did not appear to be affected by relatively high septic tank effluent COD concentrations such as those measured after approximately 30 days of operation. This suggests that the three filters were able to handle relatively strong influent, at least for short periods. Average COD concentrations were 82.7 mg/l, 65.5 mg/l, and 63.6 mg/l for systems 1, 2, and 3, respectively. Average percent reduction ranged from 88% in system 1 to 91% in system 3 (Table H-1).

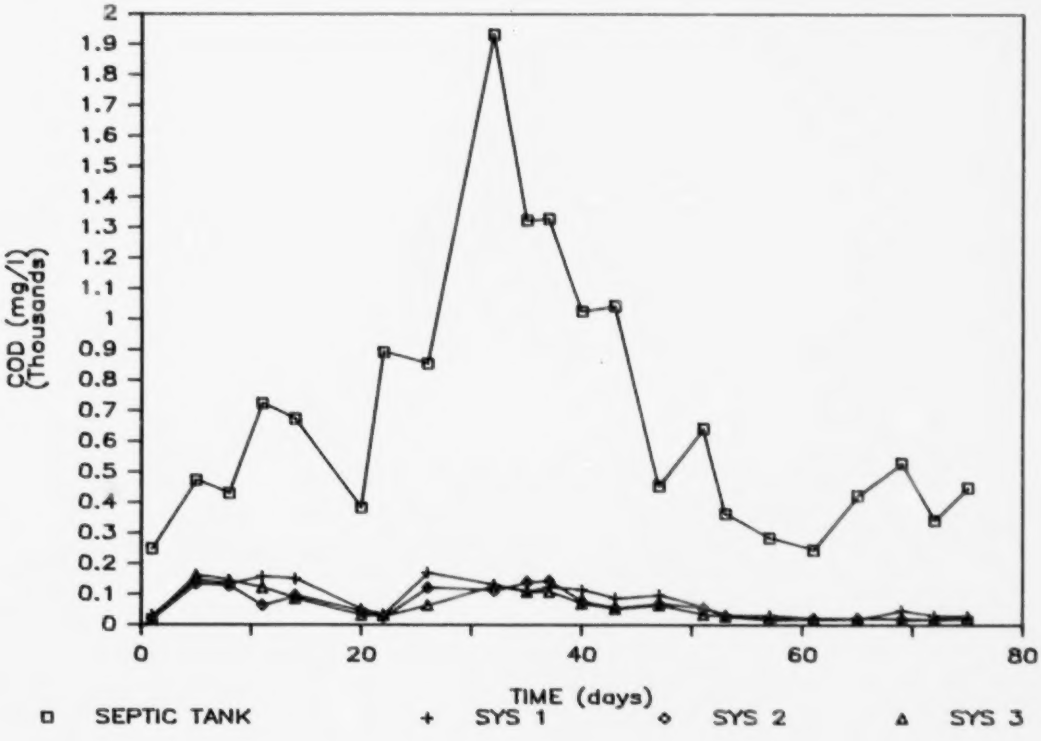

Figure H-3. COD concentrations in the recirculating sand filter columns.

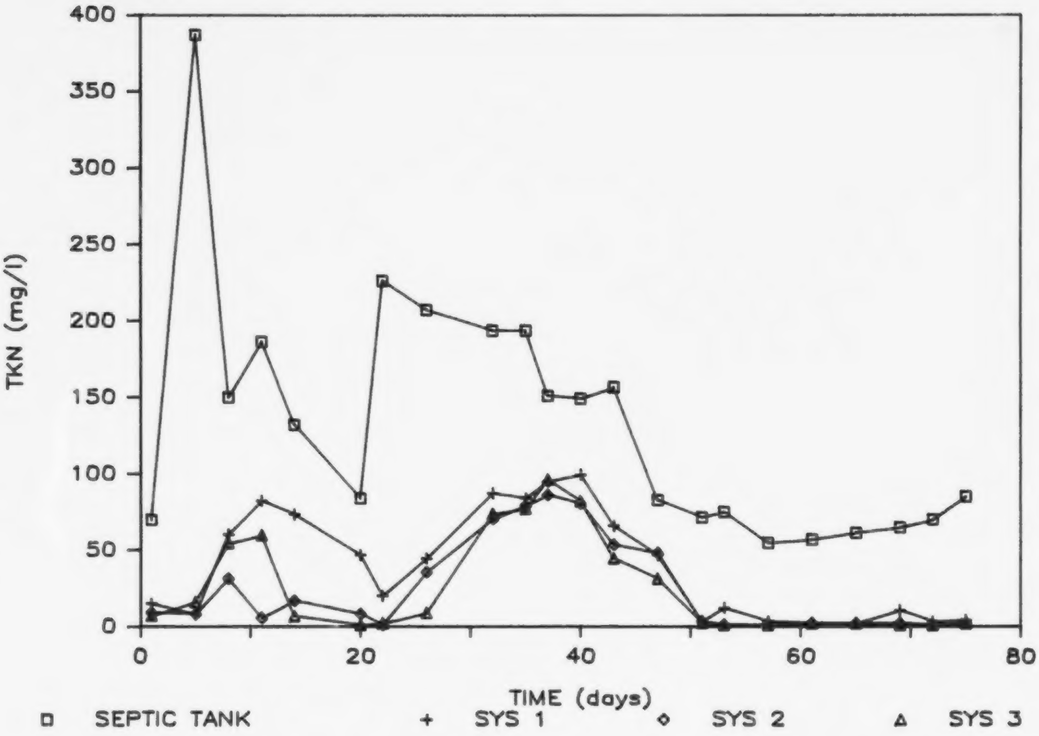

Figure H-4. TKN concentrations in the recirculating sand filter columns.

Reduction in TKN concentrations was also achieved by all three systems (Figure H-4). Early in the study, the degree of treatment was not consistent, however. System 1 frequently produced an effluent with TKN concentrations that were higher than those in the other two systems about 50% of the time. After approximately 50 days of operation, however, low TKN concentrations were consistently measured in all three RSFs. Over the study period, the average TKN concentrations were 39.7 mg/l, 25.1 mg/l, and 26.2 mg/l for systems 1, 2, and 3, respectively. Percent reduction ranged from 70% in system 1 to 88% in system 3 (Table H-1).

Total suspended solids concentrations were effectively reduced by all three systems in this study after the first week of operation (Figure H-5). The corresponding high TSS septic tank effluent concentrations during the initial phase of the study may have contributed to the higher RSF effluent TSS concentrations during that time period. Average TSS concentrations were 17.7 mg/l in system 1, 12.9 mg/l in system 2, and 13.6 mg/l in system 3 (Table H-1). Concentrations as low as 5 mg/l were common for all three systems, however. Average percent removal ranged from 88% in system 1 to 91% in system 2 (Table H-1).

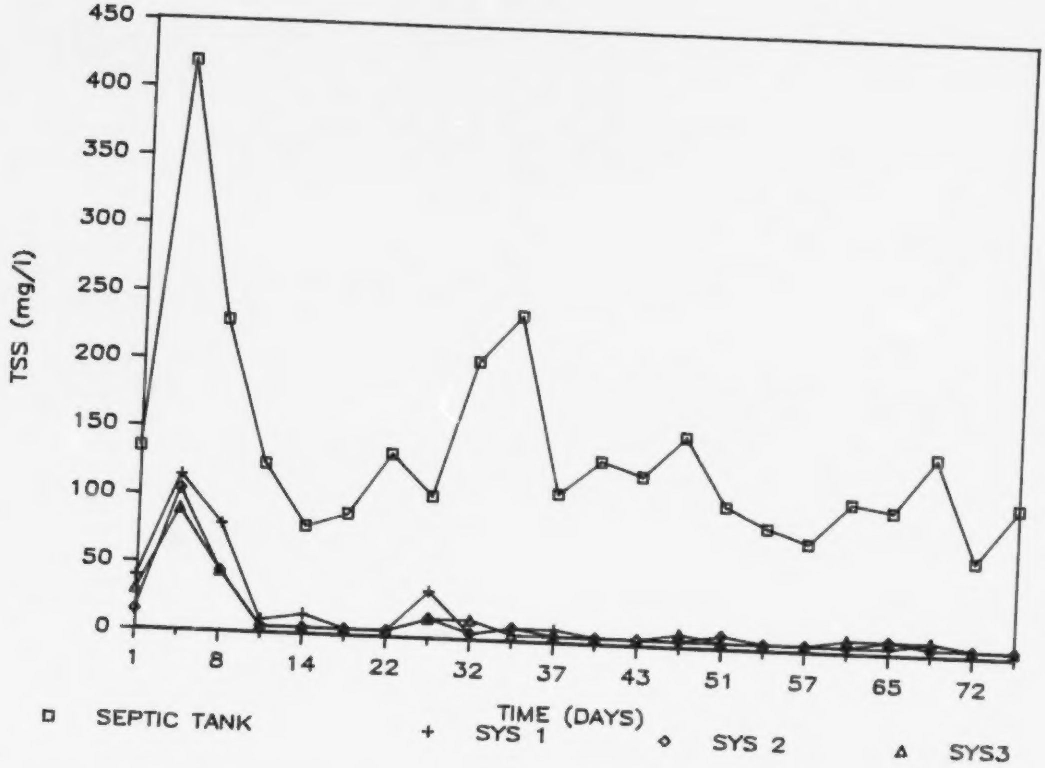

Figure H-5. TSS concentrations in the recirculating sand filter columns.

Among the three parameters measured, the greatest reduction occurred in the septic tank effluent TSS, except for the first week of operation (Figure H-5). During this period, septic tank effluent TSS was high and the filters' effluent TSS were also high. However, consistent reduction occurred across all three filters for the remaining 11 weeks.

TABLE H-1. SEPTIC TANK EFFLUENT AND RECIRCULATING SAND FILTERS EFFLUENT DATA

Parameter	Septic tank effluent	System 1	Removal (%)	System 2	Removal (%)	System 3	Removal (%)
COD (mg/l)							
Average	686	83	88	66	90	64	91
Range	246-1932	19-171		15-144		18-164	
TKN (mg/l)							
Average	132	40	70	25	81	26	80
Range	55-387	2-100		1-86		1-96	
TSS (mg/l)							
Average	142	18	88	13	91	14	90
Range	70-240	5-115		5-105		5-90	
pH							
Range	6.6-7.7	6.6-7.6		6.8-7.5		6.9-7.6	

The data were analyzed to determine whether statistically significant differences were observed for the parameters under study. Analysis of variance (ANOVA) results showed that the septic tank effluent was significantly reduced with respect to the parameters measured by all three RSFs. Thus, significant treatment was achieved by all three systems. However, statistically significant differences were not observed among the effluent TKN, COD, and TSS concentrations in the three RSFs. No difference was found in the pH measurements of the septic tank effluent and the three RSF effluents.

SUMMARY AND CONCLUSIONS

This investigation has shown that the simulated laboratory recirculating sand filter was effective in removing COD, TKN, and TSS from domestic septic tank effluent during this study.

Statistical comparison among the three systems (recirculation ratio of 3 to 1, 4 to 1, and 5 to 1) showed that they all reduced the septic tank effluent COD, TKN, and TSS equally. In this study, the difference in recirculation ratio did not influence the level of treatment achieved by the RSF. The pH measurements indicated that the pH of the septic tank effluent was not affected by passage through the recirculating sand filters.

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Appendix I

FIELD PERFORMANCE OF THREE ONSITE EFFLUENT POLISHING UNITS*

David Effert James Morand Margaret Cashell

INTRODUCTION

Soil and site conditions render many areas of the United States unsuitable for the use of conventional soil absorption field systems for onsite waste disposal. In these areas, systems that do not rely on the soil for final treatment of the wastewater are often used, many of which discharge wastewater to a watercourse for final effluent disposal. In order to reduce serious health problems or nuisance conditions that could result from the surface disposal of incompletely treated wastewater, various types of filters or other units have been used to polish pretreated wastewater prior to discharge.

Three wastewater polishing units that have been used are intermittent sand filters, upflow gravel filters, and locally, discharging subsurface gravel beds. These units are relatively small, have moderate material and construction costs, and can be easily and inexpensively maintained or rehabilitated if necessary. One of these units, the intermittent sand filter, has been shown to produce a high quality effluent (USEPA 1980), while only very limited data are available on the efficiency of upflow gravel filters or discharging subsurface gravel beds.

Two intermittent sand filters, two upflow gravel filters, and two discharging subsurface gravel beds were evaluated in this study to determine whether the effluent produced by these wastewater polishing units was of sufficiently high quality to meet or exceed surface discharge requirements for the State of Ohio. A second objective was to determine whether different levels of wastewater pretreatment, as provided by a septic tank or home aerobic treatment unit, affected the performance and reliability of the intermittent sand filters.

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* In: Proc. Fourth National Symposium on Individual and Small Community Sewage Systems. Amer. Soc. Agric. Engrs. St. Joseph, Michigan. 1984.

MATERIALS AND METHODS

Four private residences in southwestern Ohio were chosen for this study. One residence used a 4542-l septic tank followed by a pumping chamber that dosed effluent onto an intermittent sand filter. At a second residence, the wastewater was pretreated by a 4542-l Jet, Inc., home aerobic unit that was also followed by a dosing chamber and intermittent sand filter. Each of the other two residences in the study used a 4542-l Jet, Inc., aerobic unit followed by a Jet, Inc., prefabricated upflow gravel filter, followed by a discharging subsurface gravel bed. A diagram of the latter system is shown in Figure I-1.

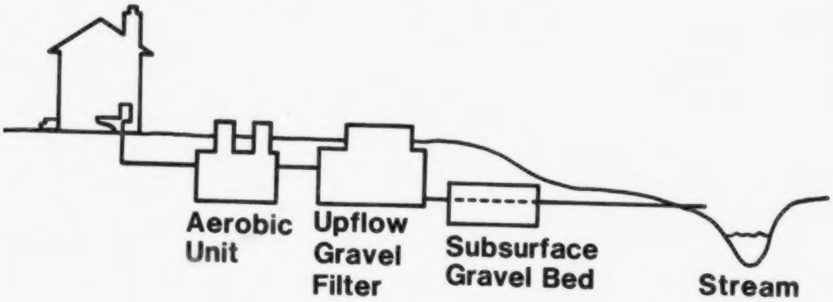

Figure I-1. Schematic flow design of a subsurface gravel bed system.

Effluent Polishing Unit Designs

The two intermittent sand filters, constructed for this study, were of a conventional design. Seventy-six centimeters of washed sand having an effective size of 0.4 mm and a uniformity coefficient of 2.5 was used as the filter media. Directly beneath the sand was 38 cm of gravel that covered the 10.2-cm diameter perforated pipe underdrain. Pretreated wastewater was dosed onto the filter using a 5.1-cm distribution pipe. The distribution pipe had two outlets that were equally spaced over the filter surface to ensure even distribution of the liquid onto the sand.

The upflow gravel filters had been operating for 3 to 3.5 yr prior to the study. The filters each had a surface area of approximately 1.24 sq m with 6.4- to 9.5-mm smooth silica stone as filter media. As wastewater from the aerobic unit flowed by gravity into the precast concrete tank that housed the filter media, it was channeled to the bottom of the tank through a riser. The wastewater then flowed upward through 15.2 cm of stone and into the discharge trough (Figure I-2). These filters are manufactured by Jet, Inc., specifically to treat aerobic unit effluent, and are necessary if the Jet aerobic system is to meet the National Sanitation Foundation Class 1 standard (Jet, Inc., 1978).

The discharging subsurface gravel beds studied received wastewater by gravity flow from the upflow gravel filters. These treatment beds are required following aerobic unit-upflow gravel filter systems in Clermont County, Ohio, and were designed by members of the Department of Health in that county. The beds were installed by excavating a 27.9-sq m hole to a level bottom depth of 76 cm. Thirty centimeters of 1.9- to 3.8-cm gravel was placed in the hole and a distribution loop of 10.2-cm diameter perforated pipe was laid above the gravel. Additional gravel was then placed around the distribution loop. A straw layer separated the gravel bed from the soil backfill. Effluent was discharged through an overflow connected to the perforated pipe (Figure I-3).

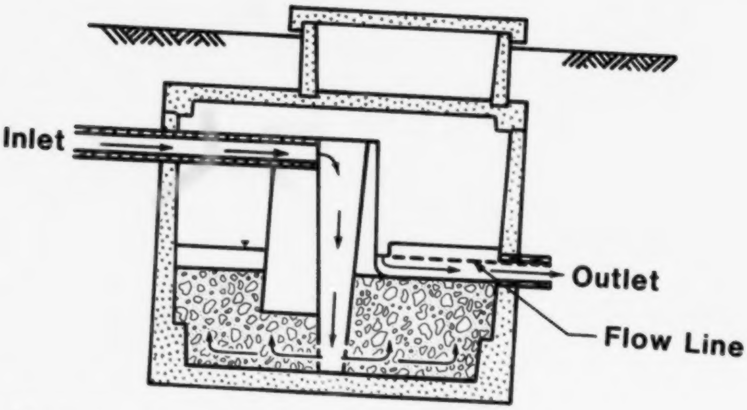
Inlet
Outlet
Flow Line

Figure I-2. Diagram of an upflow gravel filter.

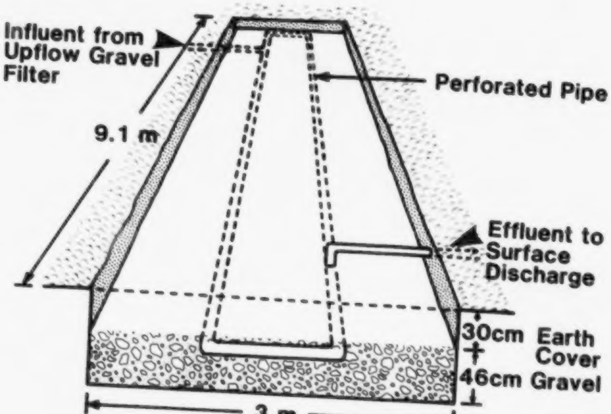
Influent from Upflow Gravel Filter
Perforated Pipe
9.1 m
Effluent to Surface Discharge
30cm Earth Cover
46cm Gravel
3 m

Figure I-3. Diagram of a typical discharging gravel bed.

Design and loading rate data for the wastewater polishing units evaluated in this study are presented in Table I-1. The two intermittent sand filters located at residences 1 and 2 were loaded at different hydraulic loading rates because of individual household water usage. The same is true for the upflow gravel filters and gravel beds at residences 3 and 4.

TABLE I-1. SYSTEM DESIGN AND LOADING RATE DATA

Item	Residence 1	Residence 2	Residence 3	Residence 4
Pretreatment	Septic tank	Aerobic unit	Aerobic unit	Aerobic unit
Effluent polishing unit	Intermittent sand filter	Intermittent sand filter	Upflow gravel filter followed by a subsurface gravel bed	Upflow gravel filter followed by a subsurface gravel bed
Surface area of filter	6.3 sq m	6.3 sq m	1.24 sq m* not valid†,§	1.24 sq m* not valid†,§
Average daily water use	945 lpd	309 lpd	709 lpd	1474 lpd
Design hydraulic loading rate	162 lpd/sq m	326 lpd/sq m	1511 lpd/sq m* not valid†,§	1511 lpd/sq m* not valid†,§
Actual hydraulic loading rate	150 lpd/sq m	49 lpd/sq m	571 lpd/sq m* not valid†,§	1188 lpd/sq m* not valid†,§
Loading method	Dosing, 3.3 times/day	Dosing, 1.2 times/day	Gravity* Gravity†	Gravity* Gravity†

* Upflow gravel filter.
† Subsurface gravel bed.
§ The loading rate cannot be determined because the loop distribution system with overflow allows effluent to move out of the bed without having to pass through the gravel.

Sampling and Analysis

The intermittent sand filters were monitored for a period of 22 months, and for the last 18 months of the monitoring program, influent and effluent composite samples were collected twice a week. The influent samples were collected during a dosing event through a piece of Tygon® tubing placed in the manifold between the submersible pump and the distribution pipe. When the pump was turned on, a portion of the wastewater was diverted to a chilled container for storage. The sand filter effluent sample was also a composite sample, collected by an automatic sampler that was activated by the dosing event. This sample was also collected in a chilled container. Typically, two dosing events were sampled during a 24-hr period to make up each composite sample.

Influent and effluent grab samples were collected twice a week for 16 weeks, typically between 9:00 and 10:00 a.m., from the upflow gravel filters and subsurface gravel beds. Samples were collected where the aerobic unit effluent entered the upflow gravel filter, in the discharge trough of the upflow gravel filter, and from the subsurface gravel bed discharge pipe. Care was taken not to disturb the biological blanket that was on top of the stone media of the upflow gravel filter.

All samples were ice-packed and transported immediately to the laboratory for analysis. Samples were analyzed for biochemical oxygen demand, suspended solids, chemical oxygen demand, fecal coliforms, dissolved oxygen, total nitrogen, nitrite plus nitrate nitrogen, and total phosphorus. These analyses were performed according to Standard Methods for the Examination of Water and Wastewater (American Public Health Association, 1980), or Methods for Chemical Analysis of Water and Wastes (USEPA, 1979).

RESULTS AND DISCUSSION

Polishing Unit Influent Characteristics

Table I-2 presents the average effluent concentrations from the four pretreatment units in this study. The septic tank effluent at residence 1 was similar in concentration to other values reported in the literature (USEPA, 1980). The aerobic units at residences 2, 3, and 4 produced effluents that were similar to concentrations found in the literature for five of the eight parameters tested (USEPA, 1980). The concentrations of total nitrogen, total phosphorus, and fecal coliform numbers were lower for all three aerobic unit effluents.

TABLE I-2. SEPTIC TANK AND AEROBIC UNIT EFFLUENT CONCENTRATIONS*

Parameter	Residence 1 Septic tank	Residence 2 Aerobic unit	Residence 3 Aerobic unit	Residence 4 Aerobic unit
BOD_5	127	28	27	31
TSS	53	38	39	62
COD	310	76	95	121
Fecal coliforms	219000	5890	32000	7900
D.O.	--	3.4	3.4	2.7
Total-N	41.5	17.8	17.3	33.3
NO_2+NO_3-N	0.6	7.4	9.3	3.7
Total-P	19.5	5.3	10.6	10.4

* All values are mean concentrations expressed as mg/l, with the exception of fecal coliforms, which are geometric means expressed as numbers of organisms per 100 ml.

Polishing Unit Effluent Characteristics

The 5-day biochemical oxygen demand (BOD_5) was consistently low in the effluents from the two intermittent sand filters, averaging 4 and 2 mg/l from the septic tank-sand filter at residence 1 and the aerobic unit-sand filter at residence 2, respectively (Table I-3). The upflow gravel filters in this study produced effluent with an average BOD_5 of 16 mg/l at residence 3, and 27 mg/l at residence 4, which was loaded at a higher hydraulic loading rate than the filter at residence 3. The discharging gravel beds further reduced the average BOD_5 to 7 and 11 mg/l for residence 3 and 4, respectively. The effluent from both the upflow gravel filters and the discharging gravel beds had a wide range of BOD_5 concentrations, indicating that the performance of these two effluent polishing units was less consistent than the sand filters.

TABLE I-3. WASTEWATER POLISHING UNIT EFFLUENT CONCENTRATIONS

Parameter	Residence 1 Septic tank Sand filter	Residence 2 Aerobic unit Sand filter	Residence 3 Aerobic unit Upflow filter	Residence 3 Aerobic unit Upflow filter Subsurface bed	Residence 4 Aerobic unit Upflow filter	Residence 4 Aerobic unit Upflow filter Subsurface bed
BOD, mg/l						
Mean	4	2	16	7	27	11
Range	2-13	1-7	1-35	2-26	7-59	2-28
TSS mg/l						
Mean	17	24	17	8	55	20
Range	1-130	2-92	2-45	3-18	13-131	3-51
COD, mg/l						
Mean	76	43	67	38	106	63
Range	4-463	12-96	36-110	11-90	68-169	40-101
Fecal coliforms, #/100 ml						
Geometric mean	1600	123	7900	1300	7900	3200
Range	13-7080000	10-91000	100-200000	100-10000	630-250000	200-250000
D.O., mg/l						
Mean	6.7	7.1	1.0	5.5	1.0	3.3
Range	4.0-9.5	5.0-8.2	0.0-2.4	3.2-7.6	0.0-2.7	2.0-4.6
Total-N, mg/l						
Mean	37.5	16.7	16.4	10.3	31.4	22.7
Range	3.2-133	8.5-32.7	5.9-30.8	5.6-16.8	0.2-47.8	17.7-37.0
NO_2+NO_3-N, mg/l						
Mean	35.3	15.8	6.9	2.2	2.5	2.1
Range	<0.1-110	4.4-32.5	0.2-21.7	0.3-10.1	0.2-10.2	0.4-10.0
Total-P, mg/l						
Mean	14.1	2.8	10.4	9.9	10.0	8.1
Range	1.8-32.6	<0.1-11.2	7.5-17.3	7.5-13.9	8.3-12.3	6.4-9.9

The probability curve in Figure I-4 was used to compare the BOD_5 values for each of the effluent polishing filters to the Ohio Administrative Code standard. The standard states that the arithmetic mean of two or more effluent samples taken at intervals of not less than 24 hr shall have a BOD_5 of less than 20 mg/l (Ohio Department of Health, 1977). Both sand filters, whether preceded by a septic tank or aerobic unit, consistently produced a high quality effluent that attained the BOD_5 standard 100% of the time. Unlike the sand filters, the upflow gravel filters produced a more variable effluent. Seventy-five percent of the individual samples from residence 3 (571 lpd/sq m loading rate) had BOD_5 values less than 20 mg/l, and 55% of the individual samples from residence 4 (1180 lpd/sq m loading rate) had BOD_5 values less than 20 mg/l. However, when the gravel filters were followed by the subsurface gravel beds, the quality of the effluent improved, and 100 and 81% of the samples collected had BOD_5 values less than 20 mg/l from residences 3 and 4, respectively.

The Ohio Administrative Code standard for suspended solids, determined by the same method described above for the BOD_5 standard, is 40 mg/l (Ohio Department of Health, 1977). The mean TSS concentration in the effluents from all of the wastewater polishing units, with the exception of the upflow gravel filter from residence 4, was below 40 mg/l (Table I-3). The range of TSS effluent concentrations was large in all of the units studied, however, and only the effluent from the subsurface bed at residence 3 had a concentration of less than 40 mg/l in 100% of the samples tested.

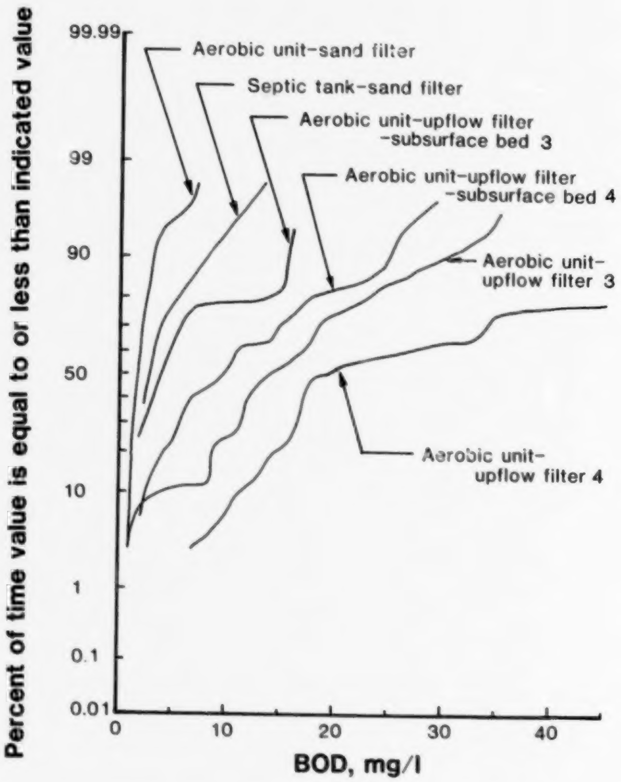

Figure I-4. Probability of level of treatment for the effluent polishing units.

The septic tank-sand filter system at residence 1 produced an effluent with a wide range of COD concentrations over the period of this study (Table I-3) and did not provide a consistent level of COD removal. However, the sand filter that received wastewater pretreated by the aerobic unit did produce an effluent with a consistently low COD concentration. This may have been because the wastewater applied to this filter had a lower and less variable COD concentration. The upflow gravel filters and the discharging gravel beds at residences 3 and 4 also produced effluent with variable COD concentrations (Table I-3), and as with BOD_5, the effluent from the discharging gravel beds had a lower average COD concentration than the upflow gravel filters.

Fecal coliforms were reduced in numbers through all of the wastewater polishing units (Table I-3). However, the mean number of fecal coliforms in all of the effluents, with the exception of the sand filter at residence 2, still exceeded the surface water standards for recreational use of 200/100 mls (Federal Water Pollution Control Administration, 1968). Therefore, it is recommended that all surface-discharged effluent be disinfected.

Both sand filters in this study consistently produced effluents with dissolved oxygen (D.O.) concentrations greater than 4.0 mg/l (Table I-3), which is an acceptable concentration for the propagation of both warm and cold water fish (Hammer, 1975). The effluents produced by either upflow gravel filter averaged only 1 mg/l and the maximum D.O. concentration was only 2.7 mg/l. The subsurface gravel bed at residence 3 discharged an effluent with an average D.O. concentration of 5.5 mg/l, while the subsurface bed at residence 4 produced an average of 3.3 mg/l D.O.

Nitrogen effluent parameters are summarized in Table I-3. As previously mentioned, the total nitrogen concentration in the effluents from all three aerobic units in this study was lower than typical values reported in the literature; therefore, effluent discharged from each respective wastewater polishing unit was also low, irrespective of the method of effluent polishing.

Nitrification was virtually complete in both of the sand filter systems, regardless of the type of pretreatment, while little nitrification occurred through either the upflow gravel filters or the subsurface gravel beds (Table I-3). The D.O. concentration of the effluents from both discharging subsurface gravel beds was relatively high, and more nitrification would have been expected. The reason for this low nitrification is not known.

There was some total phosphorus removal by the intermittent sand filters. At residence 1, a 28% reduction was seen through the filter receiving the septic tank effluent, and this performance is consistent with other sand filters studied (Sauer, 1975). A greater percentage of Total-P removal was observed through the sand filter receiving aerobic unit effluent (47%). However, the Total-P concentration in the aerobic unit effluent in this system was extremely low, and the Total-P reduction of 5.3 mg/l to 2.8 mg/l is probably not representative of the performance of a typical intermittent sand filter treating aerobic unit effluent. Little reduction of total phosphorus occurred through the upflow gravel filters or discharging gravel beds.

Pretreatment Effects

In this study, the method of pretreatment of the wastewater applied to the intermittent sand filters appeared to have little effect on the quality of the effluent produced by the filters with respect to the parameters for which there are state discharge standards. Both filters produced a high quality effluent that usually met the standards for BOD_5 and suspended solids. In addition, the dissolved oxygen concentration of the effluent from both sand filters was quite similar. Because the two filters were loaded at different hydraulic loading rates, and the individual household wastewater characteristics may have varied, it is not possible to conclude whether or not pretreatment had an effect on the performance of the sand filters for the other parameters tested.

Maintenance Requirements and Annual Operating Costs

Neither intermittent sand filter required servicing during the 22-month period that the filters were monitored. This was unexpected because intermittent sand filters having similar media characteristics and organic and hydraulic loading rates have been reported to have required some type of servicing every 6 months, including raking or replacing the first few inches of sand (USEPA, 1980). This type of maintenance is typically required to remove a crust that usually develops on the surface of the sand, because removal allows for the restoration of the sand's infiltrative capacity. A crusting layer did not develop on the intermittent sand filters during the period of the study, and as a result, no maintenance was required.

The two upflow gravel filters and subsurface gravel beds received no maintenance prior to this study, nor did they receive any maintenance during the time period of sampling, even though the manufacturer advises routine backwashing of the upflow filters at 6-month intervals. Interviews with the homeowners of these systems revealed that there had been no operational problems during the 3.5 yr that the units had been in service.

Construction costs and operation and maintenance costs for the systems are presented in Table I-4. The aerobic unit-sand filter system had the highest material and installation costs because it required both an aerobic unit and a pumping chamber. The aerobic unit-sand filter system also had the highest operation and maintenance costs, and as a result it had the highest annual costs. The septic tank-sand filter system had similar construction and annual operating costs to the aerobic unit-upflow gravel filter-discharging gravel bed systems. The least expensive system to install and operate was the aerobic unit-upflow gravel filter system without the discharging gravel bed.

SUMMARY

Both of the intermittent sand filters in this study produced a high quality effluent with respect to BOD_5 and suspended solids regardless of the type of pretreatment, and the effluents nearly consistently met the State of Ohio discharge standards. Nitrification was virtually complete and a minimum of a 28% reduction of Total-P was observed through the sand filters.

TABLE I-4. CONSTRUCTION, OPERATION AND MAINTENANCE COSTS - 1981 DOLLARS

Treatment system	Construction costs	Annual O & M*	Total annual costs†
Aerobic unit-Sand filter	$6815	188§,**	529
Aerobic unit-Upflow gravel filter-Subsurface gravel bed	3700	140§	325
Septic tank-Sand filter	3900	120**	315
Aerobic unit-Upflow filter	2900	140§	315

* Assumes pumping of the septic tank or aerobic unit every 3 yr and an additional 5 hr of maintenance per year @ $10/hr.

† Construction costs divided by a 20-yr useful life, plus annual O&M costs.

§ Assumes 550 kW-hr of electricity/yr, and aerator replacement every 10 yr.

** Assumes dosing pump replacement every 10 yr.

The upflow gravel filter receiving the lower hydraulic loading rate produced a higher quality effluent than the filter with the higher hydraulic loading rate with respect to BOD_5, suspended solids, COD, and degree of nitrification. The upflow gravel filters reduced the BOD_5 and suspended solids concentrations from the aerobic unit effluent, but the effluent from these filters did not consistently meet the Ohio Administrative Code standards for the discharging of treated wastewater off of private property.

All of the individual effluent samples from the discharging subsurface gravel beds had TSS concentrations lower than 40 mg/l, and 84% of the samples had BOD_5 concentrations less than 20 mg/l. The gravel beds caused some reduction in fecal coliforms and total phosphorus, but little nitrification occurred.

According to the Clermont County, Ohio, Department of Health, one of the potential benefits of the discharging subsurface beds is that the beds act as detention basins where additional biological treatment and the settling of suspended solids can occur. The subsurface gravel beds in this study did produce a higher quality effluent than the upflow gravel filters, but because of the method of sampling and the short duration of this study, further work would be required to determine whether or not there is enough improvement to warrant the use of the beds along with the upflow gravel filters.

The selection of an onsite wastewater treatment system that discharges effluent into a watercourse should be based primarily on system performance and reliability, and as a minimum requirement, any treatment system should consistently meet local effluent discharge standards before cost becomes an influencing factor. Both the aerobic unit-sand filter and the septic tank-sand filter systems in this study produced high quality effluents for the two parameters for which there are discharge standards. Because the septic tank-sand filter system achieved this level of treatment at a lower annual cost, the cost of installing and maintaining the aerobic unit as a pretreatment unit for the intermittent sand filter was not justified.

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APPENDIX J

A SURVEY OF EVAPOTRANSPIRATION SYSTEMS IN HAMILTON COUNTY, OHIO

Teresa Harten David Effert

INTRODUCTION

Two related methods of onsite wastewater disposal, evapotranspiration (ET) and evapotranspiration-absorption (ETA), represent potential alternatives to conventional onsite wastewater disposal. These methods utilize both the evaporation of wastewater through topsoil to the atmosphere and the transpiration of wastewater through vegetation to the atmosphere. The ETA system includes a third mechanism of disposal, the absorption of wastewater into the top and subsoil. Because evaporation and transpiration rates are dependent on climatic conditions, the success of these systems can be expected to vary with geographic location.

In 1978, the Hamilton County, Ohio, Health Department approved the use of ET and ETA systems on an experimental basis at locations where more traditional onsite treatment was not acceptable. Site restrictions such as inadequate space, impermeable soils, and steep slopes prevented the use of septic tanks and soil absorption fields at these locations. During the period from 1978 through 1981 eleven ET or ETA systems were known to be installed in the county. The performance of these systems was evaluated in 1982 and again in 1985 by staff members of the Department of Civil and Environmental Engineering at the University of Cincinnati.

MATERIALS AND METHODS

During a 3-month period from May through July 1982, and again in February through March 1985, an inventory of eleven ET/ETA onsite wastewater disposal systems was conducted to determine the effectiveness of this method of wastewater disposal. This study also attempted to identify factors contributing to the successful operation or failure of these systems. The systems were first evaluated during the summer when ET/ETA rates could be expected to be relatively high. The follow-up study was undertaken to determine how the systems functioned during the wet and cold spring period.

The information collected about each ET/ETA system was obtained through interviews with homeowners, health department personnel, plumbers and public water utilities. Onsite inspections of each system were also conducted.

ET/ETA System Design

The eleven ET/ETA systems studied were not constructed following similar design specifications. While the Hamilton County, Ohio, Health Department had issued recommendations for construction, variations in their recommendations were common in the field installations. Onsite inspections and interviews with contractors and homeowners revealed a wide range of construction practices. Figure J-1 shows a typical system, and the following is a compilation of typical design and construction practices.

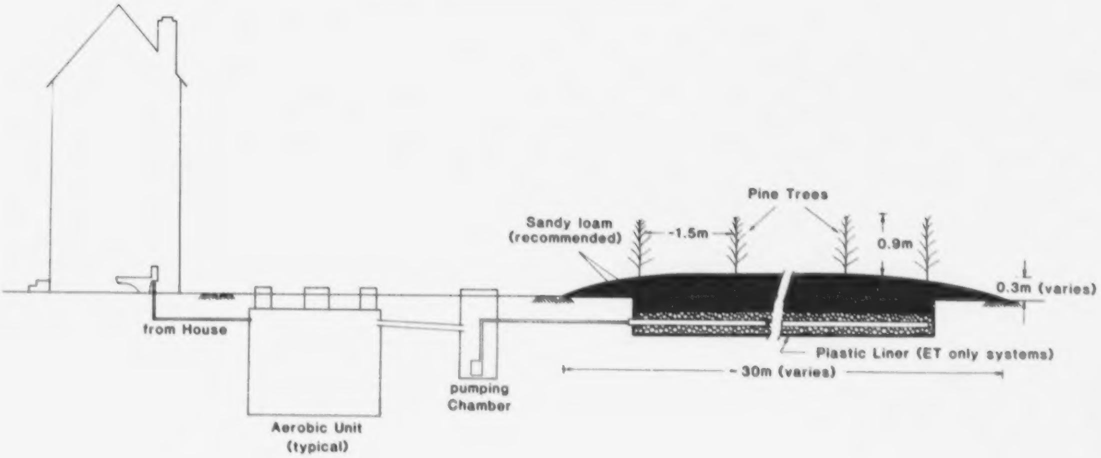

Figure J-1. Typical ET/ETA system preceded by an aerobic unit and pumping chamber.

Construction began with the excavation of a trench at least 20 cm (8 in) but up to 76 cm (2.5 ft) deep. The number of 46 to 61 cm (1.5 to 2 ft) wide trenches running longitudinally under each ET mound ranged from one to three in the systems studied, while at least one design used a bed instead of a trench.

A plastic liner was installed into the excavated trench for those systems that were to dispose of liquid solely through evapotranspiration (ET); evapotranspiration-absorption (ETA) systems remained unlined to allow for soil absorption. Next, 15 cm (6 in) of 19- to 38-mm (0.75- to 1.5-in) diameter washed gravel was laid into the trench. A 10-cm (4-in) diameter perforated PVC distribution pipe was then laid on the gravel. Another 15 to 25 cm (6 to 10 in) of gravel was added providing a total of at least 30 cm (12 in) of gravel. The gravel layer was then covered with 5 cm (2 in) of straw and the entire trench was backfilled with no more than 20 cm (8 in) of soil (Figure J-2).

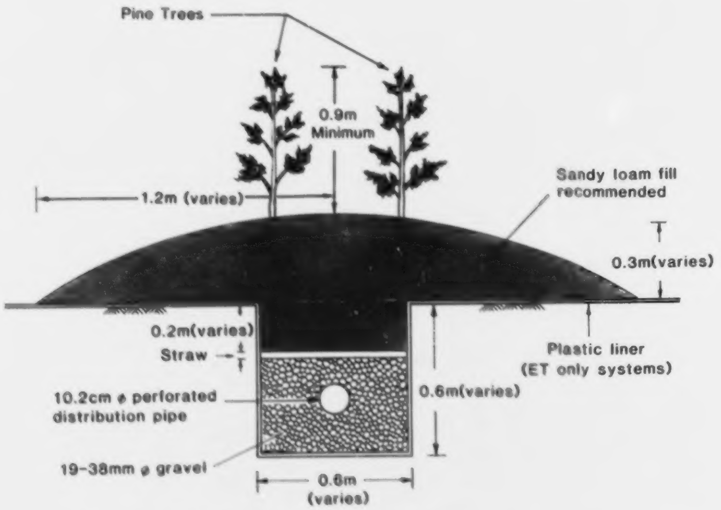
Pine Trees
0.9m Minimum
1.2m (varies)
Sandy loam fill recommended
0.3m(varies)
0.2m(varies)
Straw
Plastic liner (ET only systems)
10.2cm ø perforated distribution pipe
0.6m(varies)
19-38mm ø gravel
0.6m (varies)

Figure J-2. Cross section of a typical ET/ETA system installed in Hamilton County, Ohio.

During the final stage of construction, a mound of soil fill material was shaped above the trench. Although the Hamilton County Health Department specifies sandy loam fill, soil samples showed that heavy clays and clay loams were also used to build the mounds evaluated in this study. The final mound heights ranged from 0 to 1.2 m (0 to 4 ft) above grade with 16 or more pines, not less then 0.9 m (3 ft) tall, planted into the mound. The number of trees above each system observed in this survey ranged from 9 to 30 and consisted exclusively of Scotch and Austrian pine. The Ohio Department of Health recommends the planting of Scotch pine, Colorado spruce, or Austrian pine above the mound (Ohio Department of Health, 1977), but a study conducted by the Mid-Ohio Regional Planning Commission (1979) suggests that if trees are used to enhance ET rates, Northern white cedar should be planted because of its ability to tolerate high moisture conditions.

Except for one system that used a septic tank, treatment by a home aerobic unit preceded the evapotranspiration systems. A dosing chamber was used by all systems and wastewater was pumped from this chamber into the mounds.

RESULTS AND DISCUSSION

Onsite inspections and interviews with the homeowners were used to determine if leakage was occurring around the mound or if the homeowner had modified the system using overflows that would divert some of the wastewater away from the ET/ETA mound in an attempt to prevent seepage.

Only the sites free of breakouts and modifications were considered to be functioning properly. During the original survey conducted in the summer of 1982, six systems were determined to be in a state of failure, while the remaining five were considered to be functioning properly. During the follow-up survey conducted in the spring of 1985, the results were identical. The systems that were failing in 1982 continued to be in a state of failure, while the systems that were working in 1982 continued to function without problems in 1985. The information collected as the result of both the initial and the follow-up investigations is summarized in Table J-1 (failing systems) and Table J-2 (properly functioning systems).

TABLE J-1. DESIGN SPECIFICATIONS AND LOADING RATE HISTORY FOR THE FAILING ET/ETA SYSTEMS

Characteristic	Systems					
	1	2	3	4	5	6
Age, yr	3	4.5	6.5	6.5	6.5	6.5
Water usage, lpd	830	1120	510	610	740	750
Dimensions of mound over trenches l x w x h, m	20x3.4x0.5	33x1.5x0.2	30x3.7x0.9	33x4.3x0.9	34x4.3x0.6	30x5.2x0.4
Number of distribution lines	3	1	1	1	1	2
Liner installed	no	yes	yes	yes	yes	no
Mound fill material	sandy loam	sandy loam	clay	clay	clay	silt loam
Soil mapping unit at site*	Eden-Urban and complex	Ava-Urban land complex	Avonburg silt loam	Avonburg silt loam	Avonburg silt loam	Rossmoyne-Urban land complex
Owner modification to system	no	yes	yes	yes	yes	yes

* From Soil Survey of Hamilton County, Ohio (Lerch et al., 1982).

Homeowner Modifications

Before the initial survey was conducted in the summer of 1982, three of the homeowners concluded that the performance of their systems was unacceptable and modifications to the original design were necessary. These homeowners installed overflow pipes that diverted an unknown volume of wastewater away from their ET/ETA systems in an attempt to stop the systems from failing. By definition, these systems were considered failing at this time.

By 1985, two other homeowners had installed similar overflow pipes while a third had removed his ET/ETA system because it had never worked properly. Again, by definition, these systems were considered to be in a state of failure.

TABLE J-2. DESIGN SPECIFICATIONS AND LOADING RATE HISTORY FOR THE PROPERLY FUNCTIONING ET/ETA SYSTEMS

Characteristic	Systems				
	7	8	9	10	11[*]
Age, yr	5	6.5	7	6.5	7
Water usage, lpd	1360, est.[†]	280, est.[§]	600	800	670
Dimensions of mound over trenches l x w x h, m	27x4.9x1.2	20x-x-	30x2.6x0.3	20x4.6x0.5	10x3.7x0
Number of distribution lines	2	1	1	3	3
Liner installed	yes	yes	yes	yes/no[**]	no
Mound fill material	loam	clay	clay loam	loam	clay loam
Soil mapping unit at site[††]	Bonnell silt loam	Ava-Urban land complex	Avonburg silt loam	Rossmoyne-Urban land complex	Cincinnati silt loam
Owner modification to system	no	no	no[§§]	no	no

[*] This was the only system that disposed of septic tank effluent.
[†] Water usage estimated from 24 employees at factory, 40 hr/wk.
[§] Water usage estimated from 3 employees at service station, open 20 hr/day.
[**] Homeowner recalled the installation of a liner, but the contractor did not.
[††] From Soil Survey of Hamilton County, Ohio (Lerch et al., 1982).
[§§] Homeowner said that no modifications had been made, but neighbors thought there had been.

In 1982 and again in 1985, no modifications to the ET/ETA systems that were properly functioning were apparent, and in all cases the property owners indicated that none had been made. However, it should be noted that a homeowner may be reluctant to provide information concerning modifications made to his onsite wastewater treatment system, and all of the modifications may have not been reported.

Water Consumption

During the initial survey, water consumption appeared to be a critical factor in the successful operation or failure of the ET/ETA systems studied. Water consumption averaged 772 lpd (204 gpd) at the failing sites and 541 lpd (143 gpd) at the properly functioning sites during a period of 6 to 12 months prior to the survey. In addition, only one of the households with a properly functioning system consumed more than 757 lpd (200 gpd) while four out of the six households with failing systems consumed at least 757 lpd (200 gpd).

The 1985 survey showed that during the 2.5 yr following the initial study, water consumption patterns for the most part were unchanged in the households having the failing systems. A notable exception is household No. 2 where consumption increased. At the households with the properly functioning systems, water consumption either stayed the same or increased during the same period. On average, both groups of households consumed 750 lpd (198 gpd) by 1985. Therefore, it can be concluded that the amount of water consumption by itself did not determine the success or failure of ET/ETA systems.

Effect of the Liner

In areas where a high water table is present, liners have been used to keep the groundwater out of evapotranspiration trenches and beds (Bernhart, 1973). When ETAs are carefully lined with a material that is puncture resistant the liners may be effective, but frequently the liners are cut or torn when the gravel is placed in the trench. It could not be determined whether the liners, when present, were damaged or intact. In this study it is unclear whether the liners have a positive or negative effect on the systems because a majority of both the failing and properly functioning systems reportedly have liners.

System Age

The average age of all of the ET/ETA systems was 3 yr during the initial survey and 6 yr during the time the follow-up survey was completed. There was no apparent change in the performance of the systems as they aged, suggesting that their success or failure was independent of age at least within the range of 4 to 7 yr. Further support of this was provided by the homeowners. Those who reported their systems working noted that they had always worked; those with failing systems reported that they had never worked.

Mound Fill Material

Only two of the eleven systems surveyed used sandy loam mound fill material as recommended by the Ohio State Department of Health (1977). These two systems failed, suggesting that while proper fill may be important, its use in and of itself will not prevent failure.

Tree Cover

There was a noticeable difference in the appearance of the trees among the systems. The majority of the failing systems had trees in fair to poor condition, and only one had trees classified in good condition. By contrast, the majority of the properly functioning systems had trees in good condition and all were in fair to good condition. The trees at these locations were generally taller and fuller thus providing a better quality of vegetative cover.

Another Ohio survey on ET/ETA systems concluded that the vigor of the trees was related to landscaping and the care with which the trees were planted (Mid-Ohio Regional Planning Commission, 1979).

The report also concluded that the trees had a minimal impact on the performance of the systems, and over half of the ETA systems that had no trees were functioning properly.

The Role of Pretreatment

Only one of the 11 systems surveyed used a septic tank for pretreatment. Although this system was working properly, data are insufficient, based on this limited survey, to conclude that pretreatment has no effect on performance. Other researchers have shown, however, that the quality of wastewater has no effect on evaporation (Bennett and Linstedt, 1978).

CONCLUSIONS

The evaluation of the performance of eleven evapotranspiration systems during two distinct seasons during a 2.5-yr period has resulted in the following observations:

Fifty-five percent of the systems surveyed had never worked properly and were in a condition of failure during the time of the study. This unacceptable performance record probably reflects the overall potential of ET/ETA systems for use in Hamilton County, Ohio.

The season of the year had no effect on the performance of the ET/ETA trenches. Systems that worked well during the summer also worked well during the wetter winter/spring season. Conversely, none of the systems that failed to operate properly during the winter/spring season improved their performance and became operable during the drier summer season.

The age of the systems, within the time limits of the study, had no effect on their performance. The ET/ETA systems either worked or failed to work shortly after their installation. No improvement or deterioration in performance was seen over time.

There was no apparent relationship between the number of trees growing within the mound fill material and the success or failure of the systems. The greatest number of trees (30) growing at any single site was over a failing system, while the fewest (9) was over a properly functioning system.

Trees growing above the properly functioning systems were noticeably taller and appeared healthier than the ones growing above the failing systems. Their appearance, however, may be related to the care used during planting and the type of fill they were planted in.

Based on this limited study, the effect of the plastic liner is unclear. The hydraulic loading rates of the properly functioning plastic lined systems exceeded reported U.S. maximum evapotranspiration rates, suggesting that the liners may have been torn during construction and that absorption was also occurring.

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APPENDIX K

A COMPARISON OF PERCOLATION AND HYDRAULIC CONDUCTIVITY TESTS FOR DESIGNING SOIL ABSORPTION FIELDS

Margaret Cashell David Effert

INTRODUCTION

Soil absorption field systems are used for the treatment and disposal of wastewater in many areas that do not have access to centralized sewer systems. For these systems to function properly, the soils must absorb and transmit the wastewater at a suitable rate while treating the wastewater as it moves through the soil profile. This is partially achieved by installing a properly designed system, and this requires an accurate estimate of the rate of water movement through the soil. The percolation test, proposed by Ryon in 1926 (Federick, 1948), is currently used in many areas to provide an index of soil permeability and is required in most states to some degree to determine site and soil suitability and to design systems (Barbarick et al., 1976).

The use of the percolation test to determine soil suitability for absorption of wastewater has recently received much criticism, however. A major criticism is the wide range of guidelines within which the test can be performed. Variations exist from state to state and often from operator to operator (Hargett et al., 1983). The variables that have been found to significantly affect the percolation rate include the size and shape of the test hole (Barbarick et al., 1976; Healy & Laak, 1973; Winneberger, 1974), the initial soil moisture content and length of presoaking time (Barbarick et al., 1976; Healy & Laak, 1973; Hill, 1966), the method in which the test is performed, that is, a falling-head or constant-head test (Bouma, 1971; Hill, 1966; USEPA, 1980), and operator variability (Machmeier, 1975; Winneberger, 1974). The lack of standardized testing procedures, among other factors, results in a high variability for percolation test results (Barbarick et al., 1976; Derr et al., 1969).

Another argument against the use of the percolation test to assess the hydraulic capacity of a soil is that the relationship between the percolation rate and seepage bed performance has never been clearly established (Healy & Laak, 1973). Some feel that the percolation rate is not a measure of the effective absorption properties of a soil and should not be used as a basis for determining allowable hydraulic loads on seepage beds (Healy & Laak, 1973). Bouma (1971) concluded that the capacity of a subsurface bed to accept liquid waste cannot be adequately expressed by the percolation test because percolation rates are physically ill defined and highly variable.

Laboratory experiments on soil columns have shown that percolation rates of water into soil do not necessarily correlate with amounts of effluent that the same soil will absorb from a seepage bed because of the formation of a hydraulic resistant layer (Thomas et al., 1966). This hydraulic resistant layer, or clogging mat, forms at the infiltrative surface of a mature soil absorption field (USEPA, 1978) and decreases the rate of water movement from the field sufficiently that flow in the soil below the clogged infiltrative surface becomes unsaturated (USEPA, 1980).

As a result of clogging mat formation, many feel that the actual flow rate from a soil absorption field is governed by the unsaturated hydraulic conductivity (K) of the soil (Bouma, 1971, 1975). Hydraulic conductivity describes the ability of the soil to transmit water through pores. It expresses the rate of flow through a unit cross section in the presence of a unit potential gradient (Childs, 1969) and varies with moisture content and the resultant change in soil moisture potential (USEPA, 1980). The hydraulic conductivity of a soil decreases as the moisture content of the soil decreases. Curves of K vs. soil moisture potential can be developed for each soil, as shown in Bouma et al. (1974). Bouma (1977) and Hillel (1980a) provide further discussion of the hydraulic conductivity of a soil.

Because subsurface soil disposal of household liquid wastes involves processes of unsaturated flow, the potential of different soils for onsite disposal and treatment can only be described adequately if K-curves for the soils are available (Bouma, 1975). Bouma (1971) concluded that curves of K values for saturated and unsaturated soil were more meaningful than a single percolation rate because the curves gave the full range of possible infiltration rates (at unit hydraulic gradient) resulting from different degrees of crust resistance (clogging mat formation).

Although hydraulic conductivity values appear to provide an accurate indication of the long-term capacity of a soil to accept wastewater, there has not been much research comparing actual absorption rates to either percolation rates or to hydraulic conductivity values. In this study, saturated and unsaturated hydraulic conductivity values were obtained from various in situ tests, along with the percolation rate, and were compared to actual absorption rates. The objective of this study was to determine whether hydraulic conductivity values and percolation rates accurately described the hydraulic capacity of Avonburg and Rossmoyne soils to absorb and transmit wastewater.

MATERIALS AND METHODS

Study Area

The study site was located at the Cincinnati Center for Small Community Wastewater Systems Studies' field research site in the northern section of Clermont County, Ohio. In situ hydraulic conductivity tests and percolation tests were performed at four locations over the site. Two of the locations were in a gently sloping Rossmoyne silt loam soil (Aquic Fragiudalf, fine-silty, mixed, mesic) and two were in a nearly level Avonburg silt loam soil (Aeric Fragiaqualf, fine-silty, mixed, mesic). These soils formed in Wisconsinan loess and in the underlying weathered till of Illinoian age (Lerch et al., 1975). Soil profile descriptions from soil pits at each of the locations showed that, generally, the surface soil layer (0-10 cm) at the site was dark brown to grayish brown silt loam with granular structure. The subsurface horizon (10-25 cm) was brown to yellowish brown silt loam, with dominantly subangular blocky structure. In the Avonburg soils, mottles were present in this horizon. From a depth of approximately 25 to 50 cm was yellowish brown clay loam to silty clay loam, with subangular blocky structure. Mottles and manganese concretions were present in this horizon. A weak fragipan was present from approximately 50 to 110 cm. The fragipan was yellowish brown to brown mottled silty clay loam to clay loam, and had strong prismatic structural units that were smaller than typically present in a fragipan. Below the fragipan was mottled yellowish-brown clay loam, with massive structure and many manganese concretions. Interbedded limestone and shale bedrock occurred within 3 m over the site and was commonly found at less than 2 m. Many worm and root channels were present throughout the soil profile, and large crawfish channels were abundant over the site.

The major soil variations over the site were the depth to mottling and the expression of the fragipan. No fragipan was found at a few of the profiles investigated.

Tests Performed

The tests performed in this study included the percolation test (USEPA, 1980 - the sides of the holes were screened to prevent soil from slaking into the hole), an in situ falling-head hydraulic conductivity test (R.E. Wright Assoc., Inc. Middletown, PA.), the auger-hole method (Boersma, 1965), the instantaneous profile method (Bouma et al., 1974), and the crust test (Bouma & Denning, 1972). The crust test was also performed in the laboratory according to the procedure of Bouma et al. (1983). These methods were chosen because they are either generally accepted methods for determining hydraulic conductivity, are innovative, or are simple to perform. All of the tests give saturated hydraulic conductivity values, while the crust test and instantaneous profile method also determine unsaturated hydraulic conductivity values.

All of the tests were performed at a 50-cm depth. This depth was chosen to correspond with the depth of the infiltrative surface of a conventional system.

The percolation, falling-head, and auger-hole tests were performed at each of the four monitoring locations. The instantaneous profile method was performed at one location in a Rossmoyne soil, the field crust test at an Avonburg soil location, and the laboratory crust test on samples from an Avonburg soil location.

Actual absorption rates were obtained from two conventional trenches that were installed in an Avonburg silt loam soil. The trenches were installed at a depth of 50 cm and were 60 cm wide. They were filled with 25 cm of standard lateral gravel. The mechanism used to apply effluent to each of the trenches allowed for an accurate measurement of the volume of liquid absorbed by each trench. The ponding depth in the trench was kept constant at 17 to 23 cm. The amount of effluent absorbed per day was then calculated (Effert & Cashell, 1986).

RESULTS AND DISCUSSION

Saturated Hydraulic Conductivity Tests

The saturated hydraulic conductivity values obtained from the percolation test, the falling-head permeability test and the auger-hole method are presented in Table K-1. The data were found to be approximately log normally distributed by the use of a fractile diagram (Hillel, 1980b). Nielsen et al. (1973) also found that hydraulic conductivity data were approximately log normally distributed. Therefore, logarithmic transformations were performed on the data so that normal statistics could be applied.

The percolation test results were converted to hydraulic conductivity values by the method of Machmeier (1975). Percolation rate data should not be directly converted to hydraulic conductivity values because the hydraulic conductivity is a velocity, while the percolation rate is simply the rate of drop in water level in a given diameter hole over a period of time. Several researchers have developed mathematical relationships between percolation rates and hydraulic conductivity values (Fritton et al., 1986; Winneberger, 1974), but the equations that were developed appear to be site specific. The conversion by Machmeier (1975) was used in this study because of its simplicity and because it did not rely upon an empirically determined curve for a specific soil.

The geometric mean of saturated hydraulic conductivities, found to be a good estimator of average conductivity (Bouwer, 1969), ranged from 4 cm/day to 158 cm/day over the four test sites. A wide range of values existed in all of the tests and soils evaluated, and the untransformed data from all of the tests had relatively large coefficients of variation (CV). Although the CV of the untransformed data is not an accurate statistical parameter because the data were not normally distributed, it is useful for expressing variability on a relative basis (Hillel, 1980b) and has been used in many previous studies to assess the variability of hydraulic conductivity tests.

TABLE K-1. SATURATED HYDRAULIC CONDUCTIVITY VALUES FOR AN AVONBURG AND A ROSSMOYNE SOIL

Soil		Percolation rate (min/cm)	Hydraulic conductivity: Percolation test (cm/day)	Hydraulic conductivity: Falling-head test (cm/day)	Hydraulic conductivity: Auger-hole test (cm/day)
Avonburg 1	G*	14	28	101	12
	CV†	41%	50%	139%	33%
	n§	3	3	3	5
	Range	8.7-21	19-48	19-657	9.5-19
Avonburg 2	G	9.8	52	4	32
	CV	18%	18%	130%	70%
	n	3	3	3	5
	Range	8.7-12	42-59	1.4-18	12-82
Rossmoyne 1	G	5.8	96	19	158
	CV	47%	56%	131%	106%
	n	3	3	3	4
	Range	3.5-9.4	55-173	6.2-95	68-587
Rossmoyne 2	G	**	**	4	**
	CV			61%	
	n			3	
	Range			1.4-6.7	

* Geometric mean of the untransformed data. This is equivalent to the antilogarithm of the log-transformed data.
† Coefficient of variation of untransformed data.
§ Number of samples.
** The test was not successfully performed.

Other studies have reported large coefficients of variation for saturated hydraulic conductivity tests and percolation tests. Hillel (1980b) reported that saturated hydraulic conductivities have coefficients of variation in the 100 to 200% range. Derr et al. (1969) studied 1500 percolation tests over 250 sites and reported that coefficients of variability within the sites ranged from 0 to 253%. Bouma (1971) found coefficients of variation from falling-head percolation tests of 54%. Barbarick et al. (1976) found within-site variability to range from 7 to 48%.

Analyses of variance and Scheffe's test (SAS Institute, Inc. 1985) were used to test the logarithmic transformed data for significant differences among the three tests, and also among the four sites. No significant difference was found among the tests performed, and in terms of saturated hydraulic conductivity values, percolation rates converted to saturated K values by the method of Machmeier (1975) provided values similar to the auger-hole method and the falling-head permeability test on a site with Avonburg and Rossmoyne soils. The Rossmoyne 2 soil was significantly different from the Avonburg 1 and Rossmoyne 1 sites, but was not significantly different from the Avonburg 2 soil. However, because the Rossmoyne 2 soil had so few values, this significance has to be questioned.

Because of the lack of significant differences among the three tests and three of the soils, the hydraulic conductivity values from the Avonburg 1, Avonburg 2, and Rossmoyne 1 soils were considered to be from one population, and an average hydraulic conductivity value of 33 cm/day was obtained.

The saturated K tests and the percolation tests were relatively simple to perform. However, problems and limitations were encountered with all of them during this study. The major problem with the percolation test was with slaking of the soil during the presoaking period, and at Site 4, screening the sides of the hole did not completely prevent the slaking problem. Also, because a presoak period of 24 hr or more was used, two days of dry weather were required, and much time was lost when it rained on the second day. The percolation hole was not stable enough to hold up for a longer time period. Hauling the necessary water and gravel to remote areas to perform the percolation test could also prove prohibitive.

The auger-hole method requires no gravel or additional water, and is easy to perform. However, the hydraulic conductivities obtained with this method may differ as much as 100% between holes only a few feet apart in soil classified as a single type (Boersma, 1965), and the validity of measurements is therefore questionable. In this study, variability existed, but it was no greater than that produced by the other tests. The major problem with this test in the soils in this study was flushing the test hole out several times before performing the test, as required by the method. Because the rate of water movement was relatively slow in these soils, a long time was required for the water level in the hole to equilibrate with the water table, and this resulted in a long total testing time. Also, since these soils slake easily, the integrity of the test hole could not be maintained long enough to allow for proper flushing.

Another problem with using the auger-hole method for onsite evaluations in these soils is that the bottom of the test hole must be at least 30 cm below the water table to validate the equations used to determine the hydraulic conductivity. Because the desired depth of measurement was only 50 cm, and the water table must then be at 20 cm or shallower, it was difficult to meet this requirement. This requirement was not met at Site 4, and auger-hole hydraulic conductivities were not determined for this hole. Because a separation distance of 90 cm is usually required between the infiltrative surface and the high groundwater table, this test would not be applicable on sites that are suitable for use as onsite wastewater absorption fields.

The falling-head permeability test was easy to perform, but required bentonite, water, and gravel, all of which would be difficult to haul into a remote site.

Unsaturated Hydraulic Conductivity Tests

In the instantaneous profile method, internal drainage in a plot of soil that is artificially saturated and covered to prevent evapotranspiration is measured with the use of tensiometers and moisture content determinations (Bouma et al., 1974). From these data, the hydraulic gradient and the change in moisture content over time can be obtained, and subsequently the hydraulic conductivity can be calculated. The hydraulic conductivity at different moisture contents can be obtained by this method.

An attempt was made in this study to perform the instantaneous profile method during two field seasons, but the test was never successfully performed because the soils at the study site did not dry out sufficiently in the lower horizons, even during the driest months, to observe drainage. Another study on these soils found that very little change occurred in the soil moisture content below approximately 1 m, and that once the groundwater table receded and the macropores drained, very little water movement occurred (Cashell & Effert, 1985). It appeared that drainage occurred mainly by gravitational, or saturated, flow, and that very little capillary, or unsaturated, flow occurred. Under these conditions, the volume of water required to refill the macropores so that drainage could be observed was insufficient to produce a detectable change in the soil moisture content as measured by a neutron probe.

Therefore, it appears that the instantaneous profile method is not a suitable test to determine the hydraulic conductivity for Rossmoyne and Avonburg soils for use as soil absorption fields. Also, a long time is required to perform this test, and it requires too much monitoring equipment to be practical for onsite investigations.

The unsaturated hydraulic conductivity curve generated by the field crust test is presented in Figure K-1. The lower curve represents the combined data from three crust tests performed at one location at the study site, with a curve visually drawn through the points. The curve developed in this study is similar to curves developed in other soils with high clay contents (Bouma et al., 1974). There was a large drop in the hydraulic conductivity from nearly saturated conditions (tension = 3 cm) to more unsaturated conditions (tension = 30 cm). Assuming that the saturated K determined by the methods in this study is correct, a much larger decrease in the hydraulic conductivity occurred as the soil became unsaturated.

In this study, it was not possible to determine a hydraulic conductivity value at tensions greater than 30 cm using the most resistant crust with the crust test. The flow rate became nearly negligible at higher tensions, and the integrity of the crust could not be maintained with the long time period required to establish steady state conditions at these tensions.

The crust test was difficult to perform in the soils in this study. In order to run the test at the infiltrative surface, it was necessary to dig to a depth of approximately 85 cm to form the soil pedestal for the test (Bouma et al., 1974). As already mentioned, the soil at this depth dried out very slowly, and at approximately 1 m, remained nearly saturated throughout the year. Therefore, there was only a very short time period during which the test could be performed. Also, because of the near saturated conditions present in the lower horizons, it was difficult to keep the pit that contained the crust tests dry. The soil also has a wide capillary fringe zone, and this made it difficult to maintain high tensions in the core.

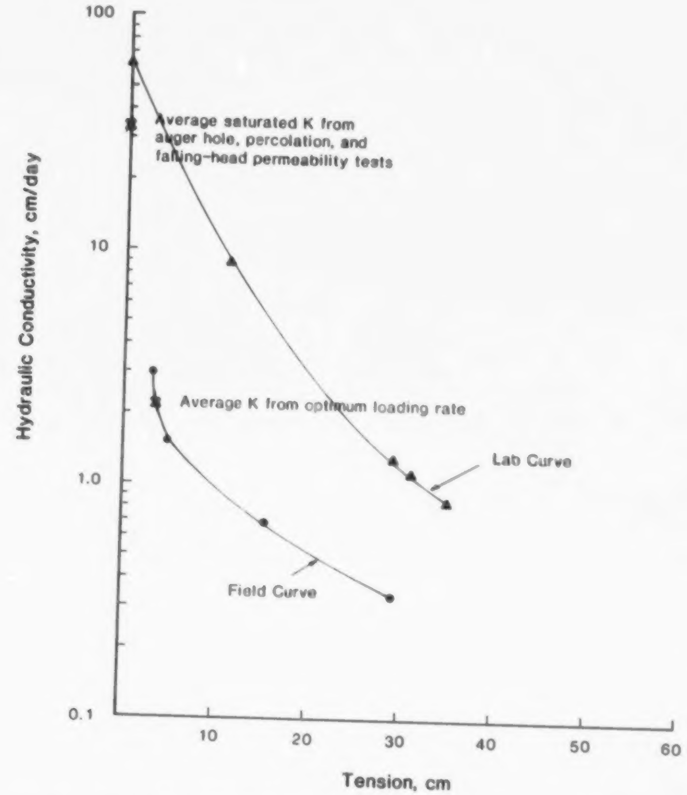

Figure K-1. Field and laboratory determined K-curves developed from the crust test.

The K-curve generated by the laboratory crust test is also presented in Figure K-1. Although this curve followed the same general shape as the K-curve derived in the field, it was about one order of magnitude higher than the field K-curve in the lower tensions, with the difference narrowing in the higher tension ranges. The saturated hydraulic conductivity obtained by this test was about twice as high as that obtained by the field saturated K tests. With a saturated K value, this difference is usually explained by the fact that a core in the lab has more macropores that may be continuous, whereas in the field, these pores may close off or become so tortuous as to greatly restrict flow. In the lab, however, because the core is not continuous with the soil below, pores may serve as open channels. The increase in the unsaturated K value cannot be explained by that reasoning, however, because the macropores are not included in the unsaturated flow regime. Other reasons, such as disturbance during removal and transportation to the lab, decreased lateral stresses because the cores were removed from the soil, or perhaps water viscosity changes as the result of temperature differences may have caused the increased K. The crust test was much easier to perform in the lab, but in the soils in this study, the curve obtained in the lab differed from that in the field, and therefore the two cannot be compared.

Although the crust test provides a great deal of information about the soil moisture regime of the soils, it is not practical to perform on a routine basis because of the length of time and amount of equipment required to perform the test. Therefore, it would not be suitable for routine onsite investigations. However, the crust test could be used to develop K-curves for the dominant soils in an area to provide additional information for site investigations.

Comparison of Hydraulic Conductivity to Actual Loading Rates

The actual average loading rates of two conventional trenches installed in an Avonburg silt loam are presented in Table K-2. The loading rates varied considerably throughout the year, being greatest during the late summer months when the water table and soil moisture contents were low and evapotranspiration (ET) rates were high, and lowest during the winter months when the water table and soil moisture content were high and ET was low. A further discussion of these loading rates is presented in Appendix D of this report.

TABLE K-2. AVERAGE DAILY ABSORPTION RATE OF CONVENTIONAL TRENCHES INSTALLED IN AN AVONBURG SILT LOAM.

	Absorption rate (cm/day)*			
Year	Jan-Mar	Apr-Jun	Jul-Sep	Oct-Dec
1983	--	--	5.9†	1.1
1984	0	1.9	2.7§	0.1
1985	1.1	1.2	1.9§	--

* Based on bottom area of trench.
† Start-up period, includes only September.
§ Includes only July and August.

The start-up loading rate of 5.8 cm/day represents near-saturated conditions, as determined by the K-curve for these soils (Figure K-1). Near-saturated conditions would be expected because a clogging mat would not have formed yet. This value, however, is only approximately 20% of the saturated hydraulic conductivity as determined by the in situ K tests performed in this study.

The mean value of loading rates from April to September 1984 and July to August 1985, 2.2 cm/day, was taken to be the long-term acceptance rate under optimum climatic conditions for these trenches. This corresponds to a soil moisture tension of 3.5 cm on the K-curve. This low tension indicates that although some resistance to flow was present in the trenches, indicating the formation of some degree of clogging mat, the soil was still in a near-saturated state. This suggests that a clogging mat developed after 2 yr did not offer a great resistance in these soils.

None of the saturated hydraulic conductivity tests or the percolation test adequately described the actual loading rates. The actual maximum hydraulic loading rate after a 2-yr period was only approximately 6% of the saturated hydraulic conductivities derived from the auger-hole method and the falling-head permeability test, and 1.4% of the percolation rate. Bouma (1971) found that real estimated flow rates were only 0.1 to 8% of the percolation rates.

When a subsurface soil absorption system is designed using the percolation test, the infiltrative surface is sized with the use of tables that correlate application rate to the percolation rate. These application rates have been empirically determined (Bouma, 1971). According to widely used recommended tables, the maximum allowable loading rate (based on bottom areas of seepage systems) for the slowest percolation rate in this study is 1.83 cm/day (18.5 lpd/m^2) (USEPA, 1980). This is similar to the long-term acceptance rate under optimum conditions for the conventional trenches in this study (2.2 cm/day). However, while the percolation rate may have provided an acceptable estimate of the actual loading rate in these soils during the summer months, the loading rates were much lower during the winter months, and the percolation rate did not provide a good estimation of the loading rates during these times of the year. Thus, the percolation test approach for sizing, although a useful tool for some situations, appears to have serious inadequacies for the soils in this study.

Use of Soil Morphology to Predict the Soil Moisture Regime

Recently, many researchers have shown that soil morphology alone can be used effectively to determine site suitability and system size, and that the percolation test, or other hydraulic conductivity tests, can be eliminated altogether. The U.S. Public Health Service produced data showing that experienced judgment compared favorably to percolation tests (Winneberger, 1974). Barbarick et al. (1976) found that the percent of sand and clay in the subsoil correlated very significantly (1%) with percolation rate, and that the percent of silt in the subsoil correlated significantly (5%) with percolation rate. Hargett et al. (1983) also found a highly significant relationship between the percolation rate and field texture data, suggesting that texture alone can be used to size systems equally or more conservatively than percolation rates. Derr et al. (1969) looked at various soil properties and found that drainage classes and the clay content of the subsoil were significantly correlated with the percolation rate, and that percolation rates varied significantly with soils developed on certain parent materials. They also found that the presence of a fragipan influenced percolation, that soils with prismatic, platy, angular blocky, and subangular blocky structure tended to have slower mean rates than soils with granular structure or soils with no structure, that loose and very friable soils tend to have higher rates than friable, firm, very firm, and extremely firm soils, and that there was a significant increase in mean percolation rates as sand content of the textural classes increased and the clay content decreased.

Some states have even eliminated the percolation test altogether as part of the site evaluation process. In North Carolina, state and local health officials can estimate the appropriate wastewater loading rate for soil absorption systems based solely on soil properties (Uebler, 1984). As of 1974, septic systems in Maine were design based solely on onsite soil evaluation (Struchtemeyer & Black, 1982). Sanitarians and engineers in Michigan are primarily using the soil profile evaluation to determine soil suitability and/or its limitations for onsite sewage disposal (Long, 1980).

Although no statistical correlations were made between soil properties and percolation rates in this study, the hydraulic capacity of the soil was inferred from the soil morphological features. The firm to extremely firm soils had silty clay loam to clay loam subsoil textures, with well-defined prismatic, subangular blocky, and angular blocky structure, and high bulk densities. Many worm and root channels were present, a fragipan or dense horizon existed, and mottling occurred at a relatively shallow depth, indicating the presence of a fluctuating high water table.

The high clay content of the soil, the firm consistency and high bulk density, the fragipan or dense horizon, and the fluctuating high water table are all properties that hinder water movement. This suggests that water movement would be slow in these soils, especially during those times of the year when the water table was high. However, the well-defined structural units and macropores suggest that flow of water could occur relatively rapidly during saturated flow.

The pattern of water movement suggested by the soil morphology was shown to be accurate by the various tests performed in this study. The percolation test and hydraulic conductivity tests showed that saturated flow occurred at a slow, but acceptable rate for wastewater absorption. However, the K-curve generated by the crust test indicated that once the macropores are removed from the drainage system, such as during unsaturated flow, the flow rate becomes very slow. The actual absorption rates showed that the high groundwater table greatly reduced the rate of wastewater absorption.

SUMMARY AND CONCLUSIONS

Hydraulic conductivity tests and percolation tests were performed at four locations over a 1.2 hectare field site containing Avonburg and Rossmoyne soils. The saturated hydraulic conductivities derived from percolation, auger-hole, and falling-head hydraulic conductivity tests were quite variable, and were not significantly different. All of these tests were relatively easy to perform, but required several days to complete.

None of the tests accurately predicted the actual absorption rate determined from conventionally installed trenches. The actual absorption rate was only 6% of the saturated hydraulic conductivity values. However, during the summer months, actual absorption rates were similar to recommended loading rates from standard tables that use percolation rates to size soil absorption fields. Actual absorption rates were much lower than the recommended loading rates during the winter months, however.

Hydraulic conductivity curves produced from the crust test method provided more information about the soils' hydraulic capacities than the saturated hydraulic conductivity tests or the percolation test. The curve showed that flow occurs very slowly in these soils once the macropores are drained. The curve also suggests that the actual wastewater absorption in these soils occurs in a near-saturated condition. Although the crust test provided much information on the hydraulic properties of the soils in this study, the field test was too complicated and time consuming to be performed on a routine basis. The laboratory test was less time consuming, but did not provide a comparable curve to the field-derived curve in this study.

The instantaneous profile method does not appear to be a suitable test to determine the K-curve of Rossmoyne and Avonburg soils for use in designing wastewater absorption fields.

Because of the large variability and low accuracy associated with hydraulic conductivity and percolation tests, it is possible that detailed soil morphological investigations by skilled soil scientists could be used to predict the hydraulic capacity of a soil as well as the various tests used to predict this. It is recommended that K-curves be developed for the dominant soils in an area, and then for each onsite investigation, only a thorough soil morphological description would be needed to accurately size the soil absorption system.

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APPENDIX L

SEASONAL VARIATION OF TWO-DIMENSIONAL FLOW FROM A WASTEWATER DISPOSAL TRENCH*

Gilbert N. Camp, Jr.

INTRODUCTION

Most onsite wastewater disposal systems have been constructed using conventional septic tank and soil absorption designs. These systems are usually sized based on observed soil characteristics and permeability testing performed at one time of the year. However, the ability of a soil to absorb effluent is variable, and most likely governed by climatological conditions during the various seasons. A system that is designed on the basis of soil tests performed during a specific season might function successfully in those given conditions, but failure may occur in a different climatic environment.

The objective of this study was to determine if climatic variables significantly affect the ability of a soil to absorb effluent from a wastewater disposal trench. A finite element analysis model was then used to simulate field conditions utilizing the hydrological characteristics of the wastewater disposal system and antecedent atmospheric and subsurface soil moisture conditions as input data.

MATERIALS AND METHODS

This study was conducted at a single family residence in Clermont County, Ohio, where a conventional septic tank-wastewater disposal system had been in service for 8 years. The site was a nearly level upland area on an Illinoian till plain, with Avonburg silt loam soils (Aeric Fragiaqualf, fine-silty, mixed, mesic). The soils developed in a silt mantle 45 to 100 cm thick and in the underlying Illinoian age weathered glacial till (Lerch et al., 1975). A similar area not used for effluent disposal was used as a control site.

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* Extended abstract from thesis. Department of Civil and Environmental Engineering, University of Cincinnati, 1985.

The soil moisture characteristic profile at the site was determined by the accumulation of data collected from a variety of installed instruments. Observation wells were used to determine the depth to the groundwater table adjacent to the trench and the depth of ponding in the trench. A depth moisture gauge (neutron probe) was used to obtain the moisture contents at various depths in the soil profile. Corresponding soil moisture tension values were obtained from manometer-type tensiometers installed to depths equal to the moisture content reading levels. Soil temperatures were determined by monitoring a series of temperature probes installed to various depths in the soil profile. These data were used to estimate the dynamic viscosity and density of the water. The location of the monitoring equipment is shown in Figure L-1.

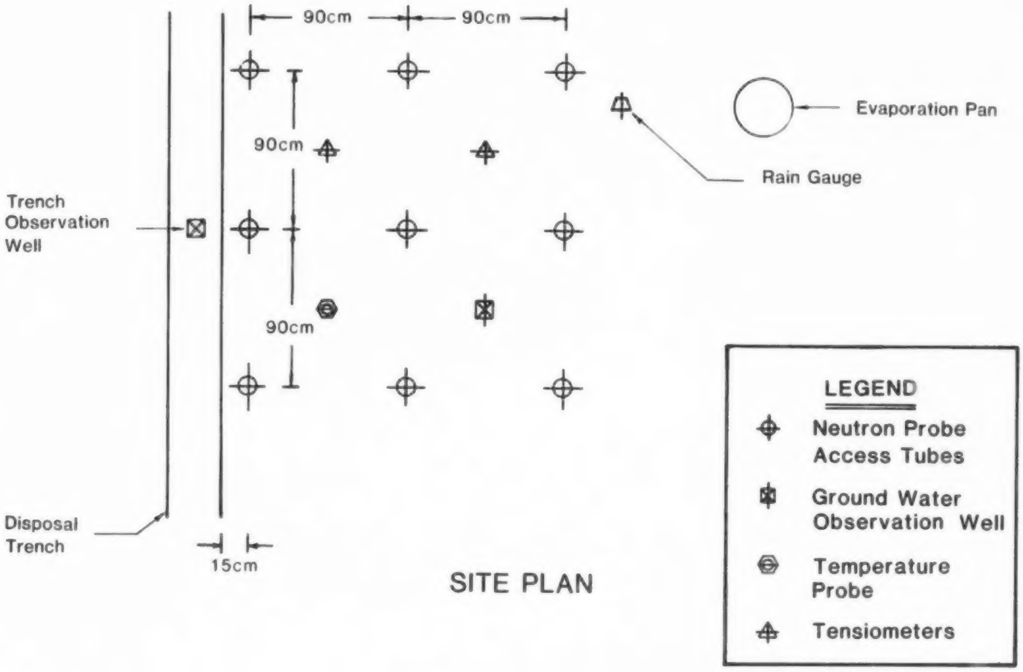

90cm
90cm
Evaporation Pan
90cm
Rain Gauge
Trench Observation Well
90cm
LEGEND
Neutron Probe Access Tubes
Ground Water Observation Well
Temperature Probe
Tensiometers
Disposal Trench
15cm
SITE PLAN

Figure L-1. Monitoring equipment location.

A tipping bucket rain gauge and a Class A evaporation pan were used to estimate rainfall and potential evaporation at the site. Soil moisture retention curves were developed in the laboratory.

The field data were used as input parameters for the FEMWATER model (Yeh & Ward, 1979). This finite element analysis model was employed to predict subsurface moisture movement from antecedent atmospheric conditions.

RESULTS AND DISCUSSION

Seasonal Variations of Soil Moisture Contents

Figure L-2 illustrates the moisture content changes that occurred from spring to summer of 1984 for Lines 1, 2, and 3, located 15 cm, 1.1 m, and 2 m, respectively, from the sidewall of the trench (Figure L-1), and for the control area. The moisture content at 30 cm below the soil surface showed the greatest seasonal variation. The dramatic variation in volumetric moisture content at this depth from spring to summer is represented by a 7% change in Line 1, 23% in Line 2, and 20% in Line 3. At a depth of 61 cm below the soil surface, there was no significant variation between the two time periods in Line 1 and only a 2 to 4% change in Lines 2 and 3. There was no change in moisture content from 1 to 1.5 m at any time during the year. In the control profile, climatological changes caused variations in the soil moisture content of the control area to a depth of approximately 0.7 m below the soil surface. A change of about 4% was seen in the soil moisture content at 122 cm between the two time periods. At a depth of 142 cm, however, no change was seen (Figure L-2).

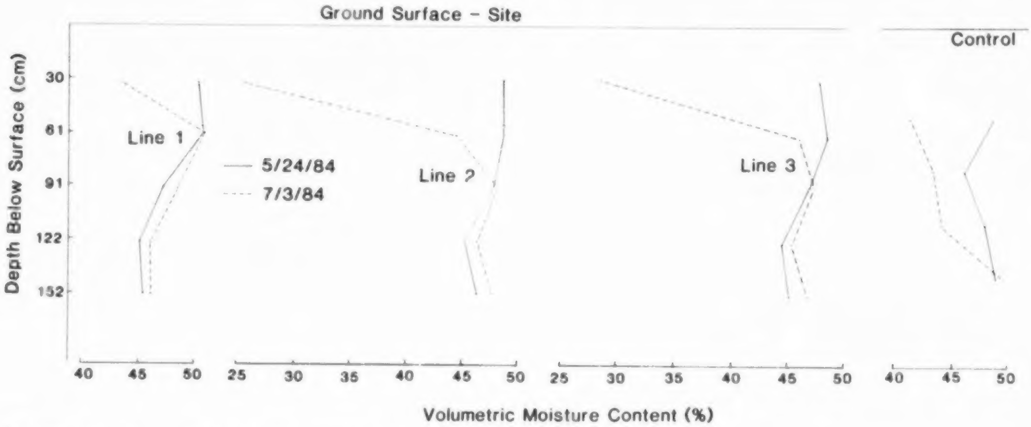


Figure L-2. Moisture content changes from spring to summer.

The minimal changes in moisture content seen in Line 1 are attributed to the closeness of this location to the wastewater disposal trench. The continual presence of water supplied by the trench minimized the external effect to the system provided by the climatological changes during different seasons. The moisture contents of the soil at the 30-cm depth in Lines 2 and 3, farther from the disposal trench, were more affected by climate. The lack of moisture variation in the lower depths of the soil profile may be the result of a high degree of capillarity in these soils and the effect of the constantly ponded trench.

Figure L-3 shows the response of the system to the weather patterns for the time period of July 4 to August 3, 1984. The changes in moisture content followed the trend of evapotranspiration and infiltration from rainfall very closely for the top 30 cm of the soil profile for the time period investigated. The reaction of the system was more dramatic for Lines 2 and 3 than the Line AT (above trench) and Line 1. Another important observation is that the moisture contents in Lines AT and 1 were always greater than the other two lines because of their locations relative to the trench. A slight time lag occurred at the 30-cm depth during midsummer with the increase in moisture content caused by a rainfall following an extended dry period. The soil moisture continued to increase 4 days after the termination of rain.

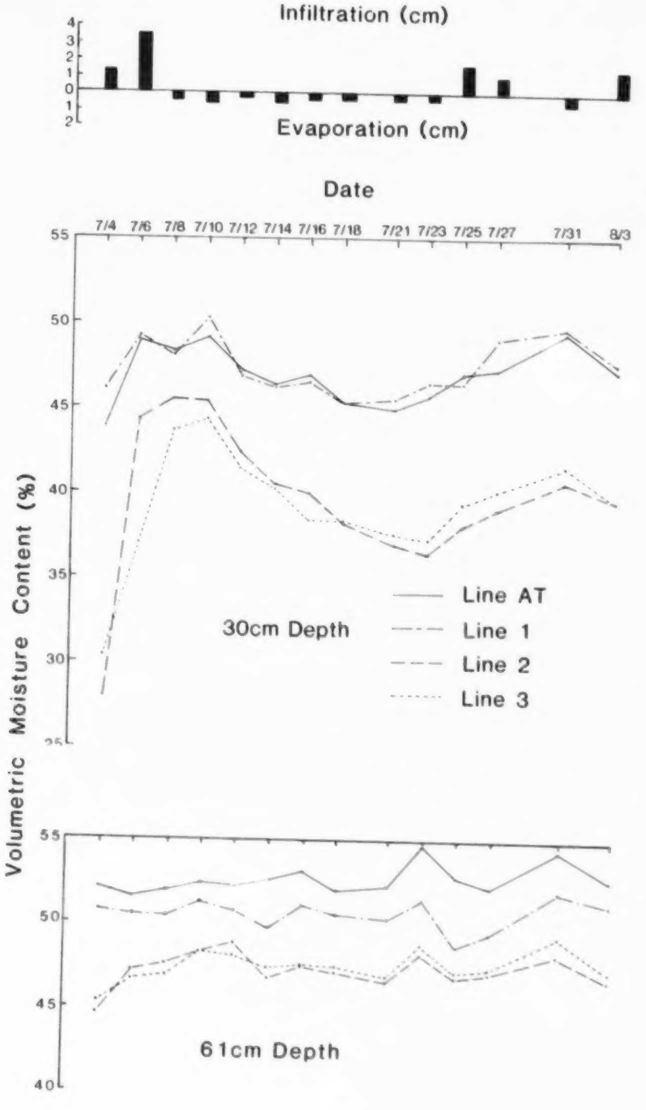


Figure L-3. Seasonal variation of moisture content July 4, 1984 to August 3, 1984.

At the 61-cm depth the variations were much less pronounced than at 30 cm. During the spring, there was almost no change in moisture contents at this depth because of the continual high degree of saturation maintained by the greater rainfall. The changes in soil moisture seldom followed the infiltration and evapotranspiration events during the summer.

Fluctuations of the Groundwater Table

The groundwater table fluctuated with rainfall events in this study. Figure L-4 shows the reaction of the groundwater table to the infiltration and evapotranspiration events from July 4 to August 3, 1984. The magnitude of fluctuation of the groundwater table within the system was great. After a relatively dry period in which the groundwater table was low, approximately 5 cm of infiltration resulted in an observed rise of 70 cm in the groundwater table from July 3 to 6, 1984. Evapotranspiration then contributed to the 50-cm drop after that rainfall. Following the rapid decline in the water table from July 6 to 10, a more gradual decrease in the water table occurred during a period of continuous evapotranspiration, with minor fluctuations occurring with rainfall events.

One reason for this rapid change in the groundwater table is the high degree of capillarity of the fine-grained soils in this study. As previously mentioned, there was very little change in the moisture content of the lower horizons of these soils during the year. Therefore, only a small volume of water was needed to provide continuity of the fluid within the system, resulting in saturated conditions. Once the macropores drain, the water in the pores is no longer continuous, and a water table is not detected. Cashell and Effert (1985) also found this on similar soils.

Soil Moisture Potential

Tensiometers were installed in two locations at depths of 30, 91, and 152 cm, as shown in Figure L-1, to determine the soil moisture potential at these locations. These data were used to establish the saturated and unsaturated zones of the soil. Figure L-5 shows the tensiometer data for the same time period as in Figures L-3 and L-4. Groups 1 and 2 represent the tensiometers located 60 and 150 cm from the trench, respectively.

During the time period shown in Figure L-5, the tensiometers at the 30-cm depth in both groups showed nearly continuous unsaturated conditions (negative matric potentials) with varying degrees of moisture depending on climatological events. At the 91-cm depth, the tensiometers in Group 1 showed saturated conditions existing (positive potential), while in Group 2, located farther from the trench than Group 1, saturated to nearly saturated conditions existed. The only time that the tensiometers at this depth showed nearly saturated rather than saturated conditions was when the groundwater table was greater than 12 cm below the point being measured by the tensiometer. Although the matric potential should be negative, showing unsaturated soil, when the groundwater is below the point measured, there were many times during this study when positive potentials existed. The reason for this was not investigated, although a similar reaction was seen in similar soils (Cashell and Effert, 1985). The soil moisture potential was positive through the time period shown in Figure L-5 for the 152-cm depth, which was expected since the groundwater table was above this depth throughout the time period (Figure L-4).

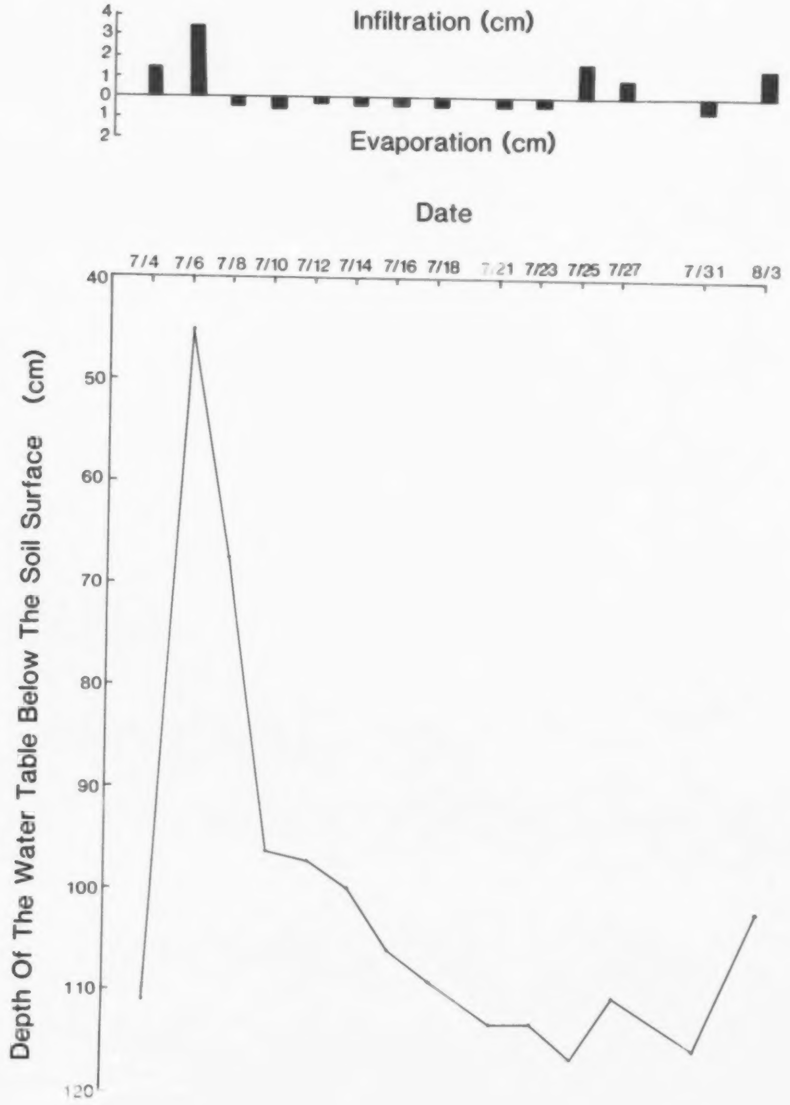


Figure L-4. Variation in the groundwater table July 4, 1984 to August 3, 1984.

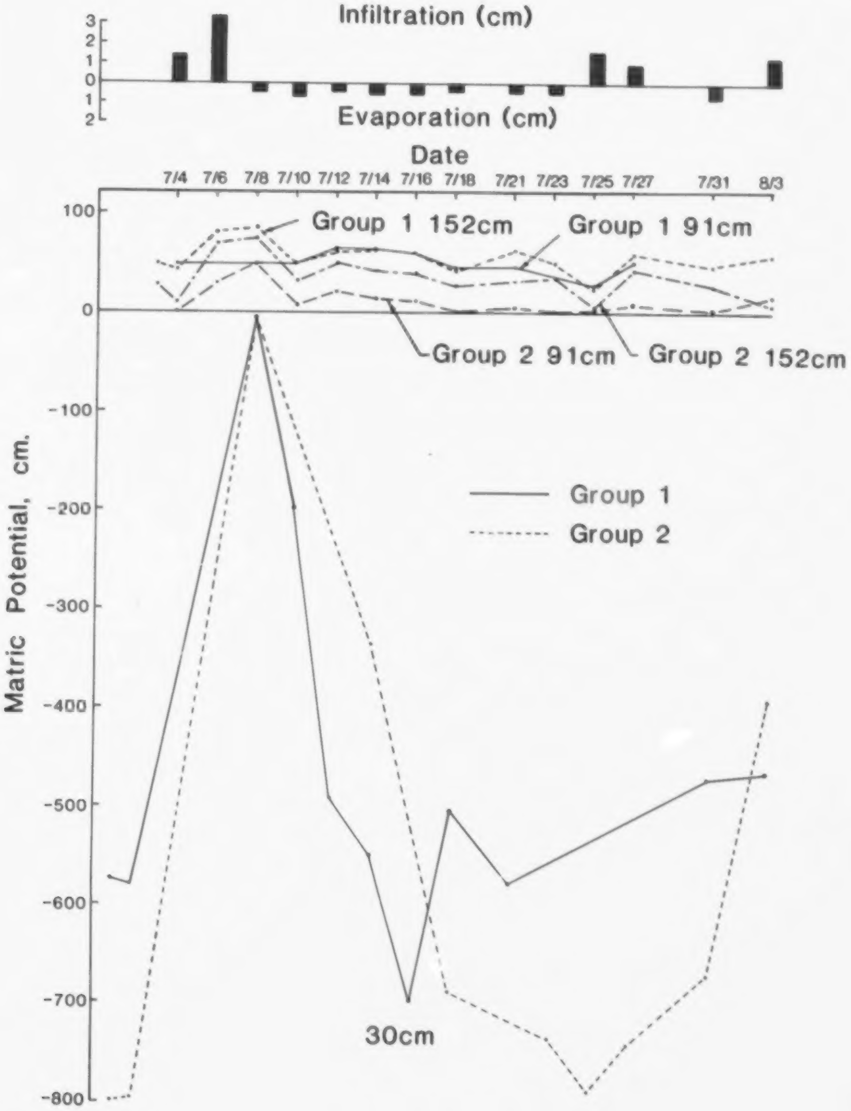


Figure L-5. The matric potential at various depths from July 4, 1984 to August 3, 1984.

Estimation of Field Conditions by the Finite Element Model

The FEMWATER model (Yeh & Ward, 1979) was used to predict the reaction of the wastewater disposal system to given antecedent atmospheric and subsurface soil moisture conditions. The computer results were then compared with actual field data to determine the accuracy of the model and input parameters.

The investigation of the system using the model spanned a period of 6 days with field measurements taken at time = 0, 3, 4, and 6 days. Field data obtained at time = 0 was used as initial input conditions for the model. Field moisture content measurements were compared with the laboratory moisture retention curve to provide initial pressure heads at specific nodes within the soil profile.

For the simulation, the region of interest was divided into 105 nodes and 84 quadrilateral elements. The boundary conditions consisted of two Dirichlet profiles (16 nodes) and one Cauchy profile (9 nodes, 8 element sides). The Dirichlet boundaries were chosen at the trench-soil interface and the bottom row of the element grid. Values for the prescribed heads were obtained from field measurements obtained from the observation wells located in the disposal trench and in situ soil, and minor fluctuations in the well levels were incorporated in the Dirichlet profiles at the proper times. The Cauchy boundary was established at the soil surface to allow surface moisture fluxes and possible ponding.

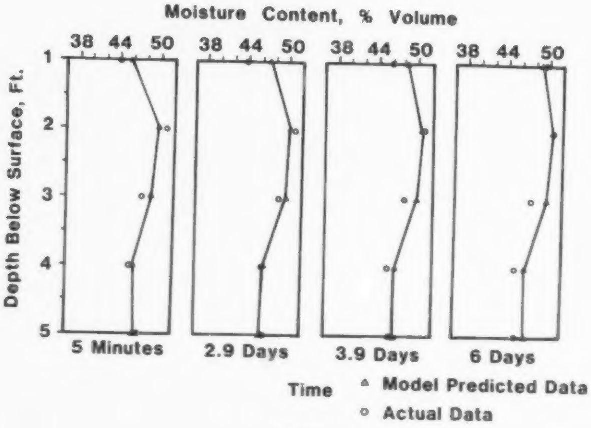
The transient analysis was obtained from mass matrix approximation for 55 time steps, starting with 5 min and increasing to 3-hr increments. The soil surface was subject to a constant evaporative force up to 3 days, after which rain produced an infiltrative flux of 1.4×10^{-4} $cm^3/sec/cm^2$ at 6 days. The variations in all the region boundary profiles provided the forces required for a dynamic system.

Field measurements obtained at the relevant times were compared with the modeled predictions, and the calculated and field moisture contents are shown in Figure L-6. The moisture contents generated during the simulation showed trends in volumetric changes similar to the field data. The greatest reactions occurred at the soil surface and steadily decreased in dynamic amplitude as the location of data approached the groundwater table. A maximum moisture content difference of 10 % occurred between field water contents and computed values.

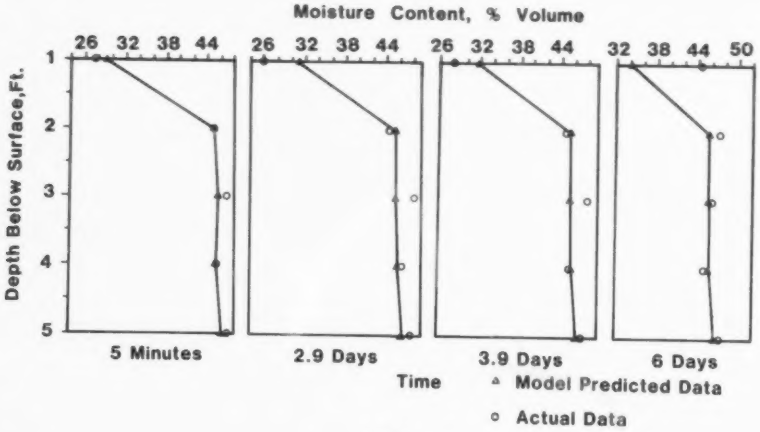
Probable causes for the variances between the field data and the predicted data lie both in field measurements and input variables. Input pressure heads were accurate at points of obtained data within the soil profile, but had to be interpolated elsewhere. Also soil sample manipulation has long been recognized as a source of data error.

The divergence from field to modeled moisture contents in the unsaturated zone is a result of estimated relative permeabilities and Cauchy flux rates at the surface boundary. Below the groundwater table the minor errors may have resulted from variations in entrapped air volumes.

LINE 1



LINE 2



LINE 3

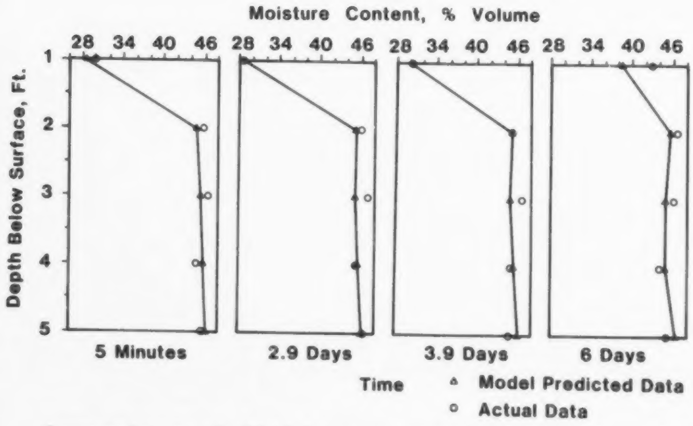


Figure L-6. Comparison of field measurements of soil moisture content to modeled values.

SUMMARY AND CONCLUSIONS

The soil moisture contents, location of the groundwater table, and soil temperature all influence the ability of the soil to accept, transport, and purify effluent from a wastewater disposal trench. Rainfall and evapotranspiration are the two most important factors controlling the parameters, as shown by the daily variations of the system in this study. At the site investigated for this study, the high moisture contents and high groundwater table during the winter and spring resulted in seasonal failure of the system because the soils could not absorb effluent. However, during the summer and fall, the soil moisture content was lower and the groundwater table was deeper, and the soil was able to dispose of the wastewater easily.

In the trenches in this study, the water from the trench had a lateral influence of at least 2 m from the trench sidewall. Therefore, the recommended trench spacing of 1.8 m (USEPA, 1980) may not be sufficient in soils such as those in this study to optimally dispose of the effluent.

Computer modeling of saturated and unsaturated flow is a viable option as a method for subsurface disposal system design. With sophisticated laboratory tests and minimal field data used as input variables, the FEMWATER computer code can accurately determine field conditions. However, it should be cautioned that a model is only as good as the input data and careless use of the model will produce erroneous results.

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Lerch, N. K., W. F. Hale, and E. L. Milliron. 1975. Soil Survey of Clermont County, Ohio. Soil Conservation Service, United States Department of Agriculture, Washington, D.C. 139 pp.

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Yeh, G. T. and D. S. Ward. 1979. FEMWATER: A Finite Element Model of Water Flow Through Saturated-Unsaturated Porous Media. ORNL-5567, Oak Ridge National Laboratory, Oak Ridge, Tennessee. 162 pp.

APPENDIX M

QUALITY ASSURANCE/QUALITY CONTROL

INTRODUCTION

This section presents a summary of the quality assurance/quality control data from this project.

Throughout this project, a quality assurance/quality control plan was followed for sample analysis to ensure that the results were the best possible estimations of true concentrations. The three main quality assurance objectives for measurement data were precision, bias (accuracy), and completeness. These terms are defined in USEPA, 1984. The quality assurance objectives in this study for the parameters measured are given in Table M-1.

TABLE M-1. QUALITY ASSURANCE OBJECTIVES

Measurement	Reference	Precision (% CV)	Accuracy	Completeness
COD	410.4 (2)	±14%	±10%	
BOD_5	507.0 (1)	±14%	±10%	90%
TSS	209 D (1)	±28%		90%
Total-P	365.4 (2)	±14%	±10%	
TKN	351.2 (2)	±14%	±10%	
NH_3-N	350.1 (2)	±14%	±10%	
NO_2+NO_3-N	353.1 (2)	±14%	±10%	
NO_2^--N	353.1 (2)	±14%	±10%	
Fecal coliforms	908 C (1)	±26%		90%
pH	423.0 (1)	±0.1 pH limit		

In this project, sample analysis was performed at two different laboratories. The analyses for fecal coliforms, total suspended solids (TSS), pH, dissolved oxygen, and biochemical oxygen demand (BOD_5) were performed in the Department of Civil and Environmental Engineering at the University of Cincinnati. The analyses for chemical oxygen demand (COD), ammonia (NH_3-N), total Kjeldahl nitrogen (TKN), nitrate and nitrite (NO_3+NO_2-N), nitrite (NO_2-N), and total phosphorus (Total-P) were conducted by the United States Environmental Protection Agency's

Environmental Monitoring and Support Laboratory, Cincinnati, Ohio. The analyses were performed according to the methods indicated in Table M-1. The sampling schedules for the various studies are given in the Appendices.

The quality assurance objectives for this project were met by performing duplicate analyses on at least 5% of the samples to obtain precision, and spiked samples and standards to obtain accuracy, where applicable. For the samples analyzed at the University of Cincinnati, duplicate sample analysis was routinely performed. Accuracy for pH and BOD_5 was evaluated by analyzing samples of known composition prepared by an outside laboratory. This was part of the EPA's Wastewater Pollution Laboratory Performance Evaluation, and samples were run semiannually. Accuracy for total suspended solids was not determined because there is no satisfactory procedure for obtaining the accuracy of the method on wastewater samples because the true concentration of suspended matter is unknown. Accuracy for fecal coliforms was also not determined.

For the samples analyzed by the EPA laboratory, precision and accuracy were determined internally by that laboratory using both spiked and standard samples. Also, approximately once a month, duplicate samples, spiked samples, and standard samples were also included with the usual samples that were sent to the EPA laboratory to assess the precision and accuracy of that laboratory. The precision and accuracy results reported below are taken from the EPA's quality assurance reports during the time period of the analyses. No completeness data are available from these reports.

CHEMICAL OXYGEN DEMAND

Precision

Chemical oxygen demand precision is shown in Figure M-1 for the influent (primary effluent) and effluent samples. All of the influent duplicate samples had percent coefficients of variation (CVs) that were less than 14%. This shows a high reproducibility or precision for the COD analysis performed on the influent samples. A poorer precision was found for the effluent samples where only about one-half of the duplicates had CVs less than 14%. However, for effluent concentrations of the same order as the influent, the relative errors were indeed less than 14%.

Bias

Bias in the COD test was based on spiked sample recovery for influent, effluent, and groundwater as shown in Figure M-2. For influent the mean recovery was 108% with a standard deviation of 21.5%. With the large value of standard deviation, this recovery is not different from 100%, although the standard deviation is larger than is acceptable in Table M-1 of 5%. The same is true for effluent and groundwater samples where the mean

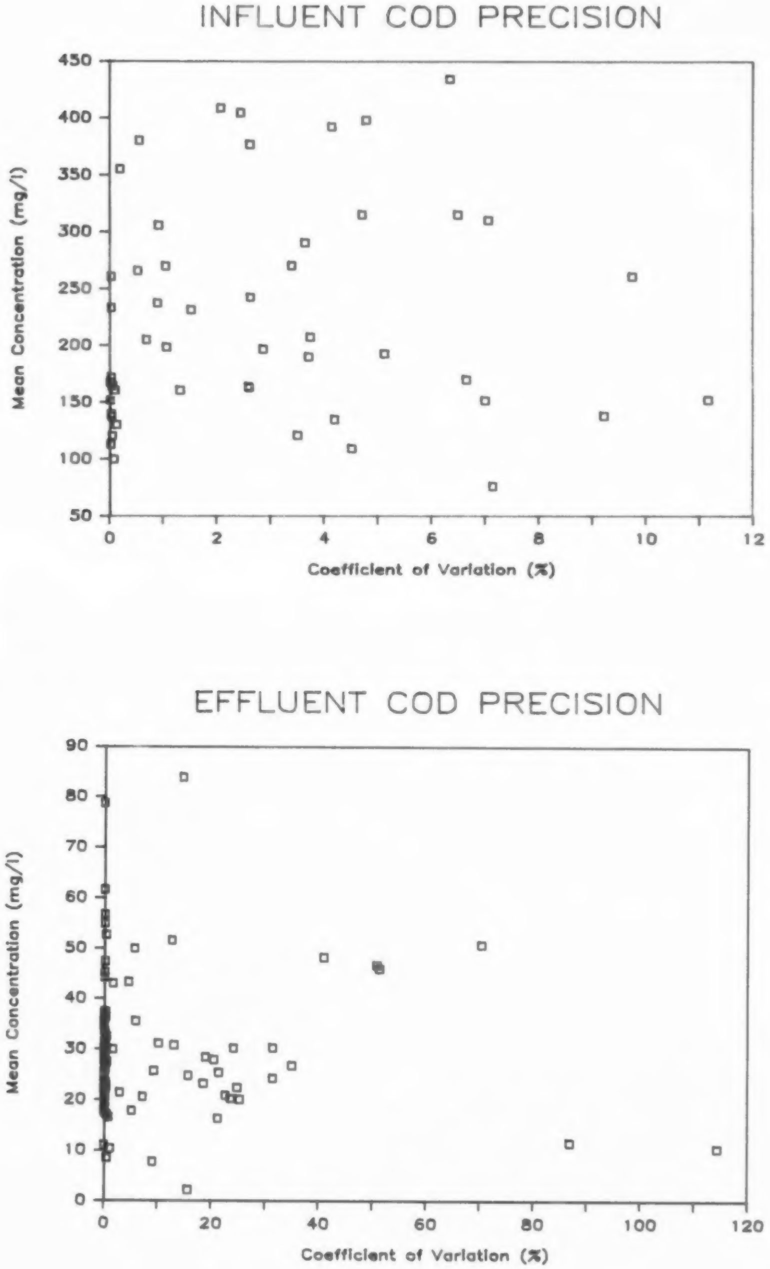


Figure M-1. COD precision.

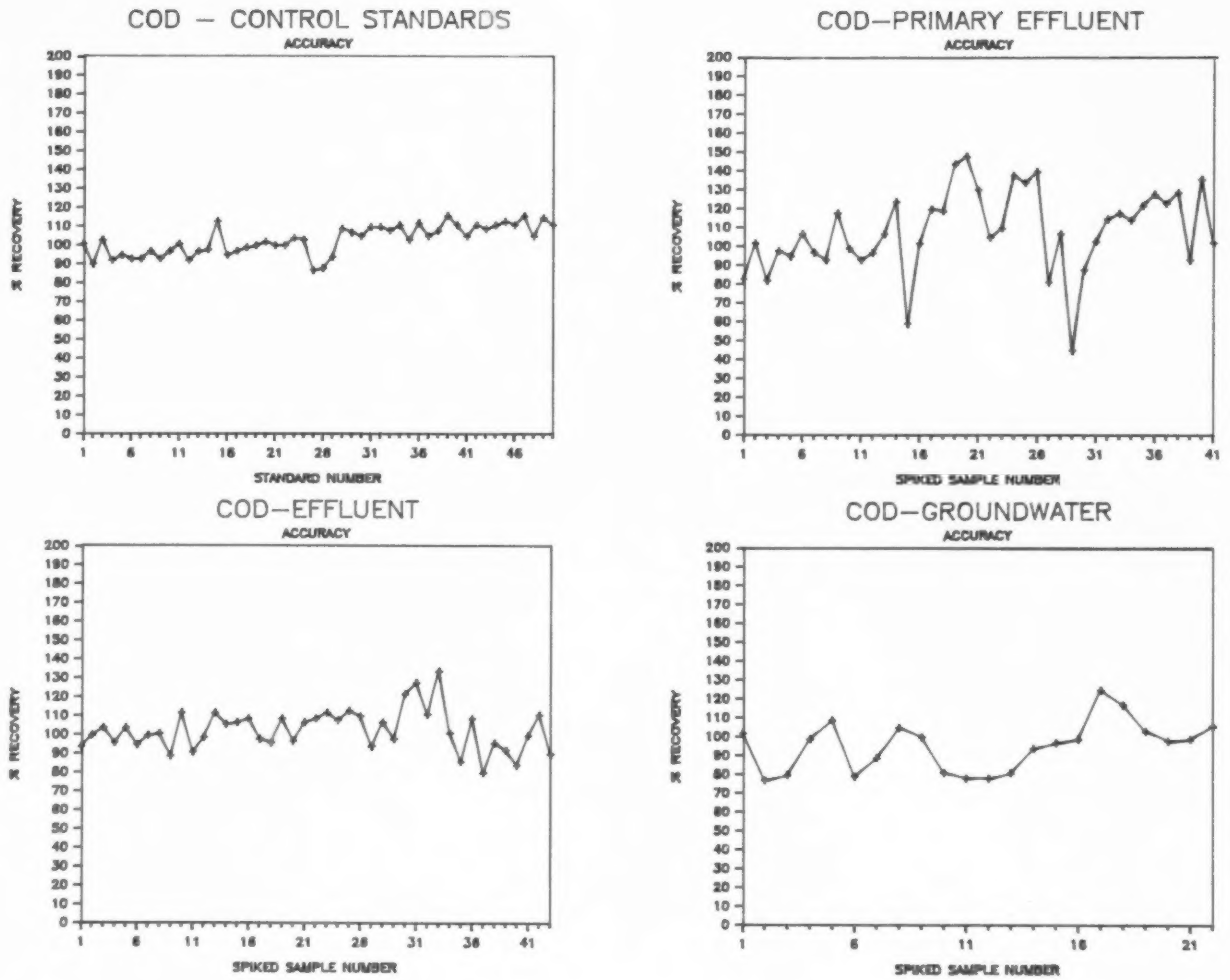


Figure M-2. COD accuracy.

recoveries are 102.8 and 95.3 and the standard deviations are 10.8 and 13.2, respectively. Control standards show a 102.8% recovery with a standard deviation of 7.8%, closer to that suggested in Table M-1. These values are deemed acceptable for the purposes of this report.

BIOCHEMICAL OXYGEN DEMAND

Precision

The precision of the BOD_5 test is depicted in Figure M-3. The pooled CV is 13.3%, which agrees with the value of 14% shown in Table M-1. About 80% of the duplicates were below 14% CV and 97% were less than 28%. These results are deemed acceptable.

Bias

No tests for bias were done for the BOD_5 measurements. Test samples sent from EPA to the University of Cincinnati lab were analyzed for BOD_5 and 6 out of 8 measurements were within the control limits (Table M-2). This showed that the accuracy of the test procedure was adequate.

TABLE M-2. PERFORMANCE EVALUATION SUMMARY OF BOD_5

Date	Report value	True value	Acceptance limits	Warning limits	Performance evaluation
12/3/82	4.7	2.51	.704-5.03	1.24-4.49	Warning.
	49.0	154	93.7-204	107-190	Not accepted.
5/27/83	44.9	77.3	48.5-104	55.3-97.1	Not accepted.
	73.6	109	63.2-147	73.5-136	Accepted.
12/14/83	3.51	3.42	1.42-5.42	1.91-4.93	Accepted.
	128	129	85.3-174	96.2-163	Accepted.
5/24/84	26.6	31.1	18.3-42.9	21.9-39.8	Accepted.
	44.7	52.1	31.3-71.1	36.3-66.1	Accepted.

Completeness

The average completeness for BOD_5 measurements was 80% calculated as the ratio of the average number of useful data points to the number of samples collected, not including quality assurance duplicates. This value is heavily influenced by one sampling period during which only 60% completeness was achieved. Although not as complete as the desired 90% indicated in Table M-1, the data collected were adequate to characterize system performance.

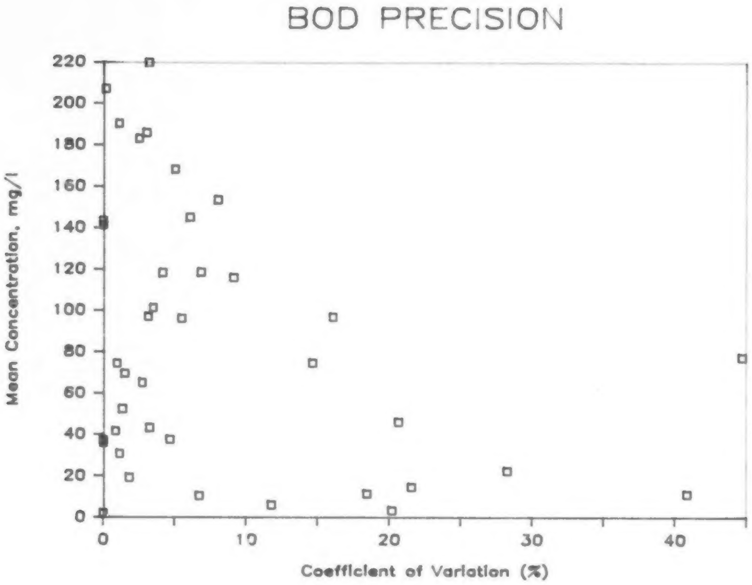


Figure M-3. BOD$_5$ precision.

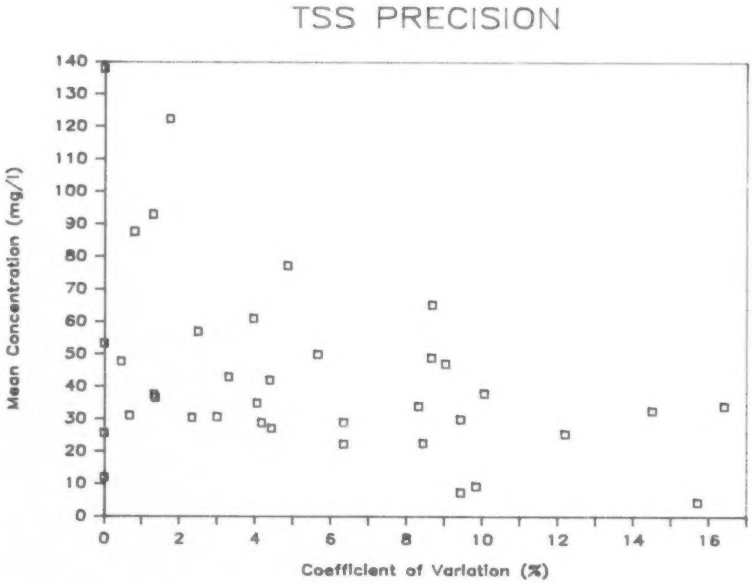


Figure M-4. TSS precision.

TOTAL SUSPENDED SOLIDS

Precision

The TSS precision is shown in Figure M-4. The pooled CV is 7.1% with 93% of the duplicates less than 14% and all less than 17%. These results compare favorably with the value of 28% listed in Table M-1.

Bias

No standard exists to assess bias for TSS measurements in wastewater samples.

Completeness

The TSS data were 96.9% complete, well within the guidelines in Table M-1.

TOTAL PHOSPHORUS

Precision

The precision results for Total-P are plotted in Figures M-5 for influent and effluent, respectively. The pooled CV is 3.5%, which compares favorably with the value of 14% listed in Table M-1. Ninety-seven percent of the duplicates had CVs below 14%, while all values were below 28%. These results are deemed to be of acceptable analytical precision for both influent and effluent samples.

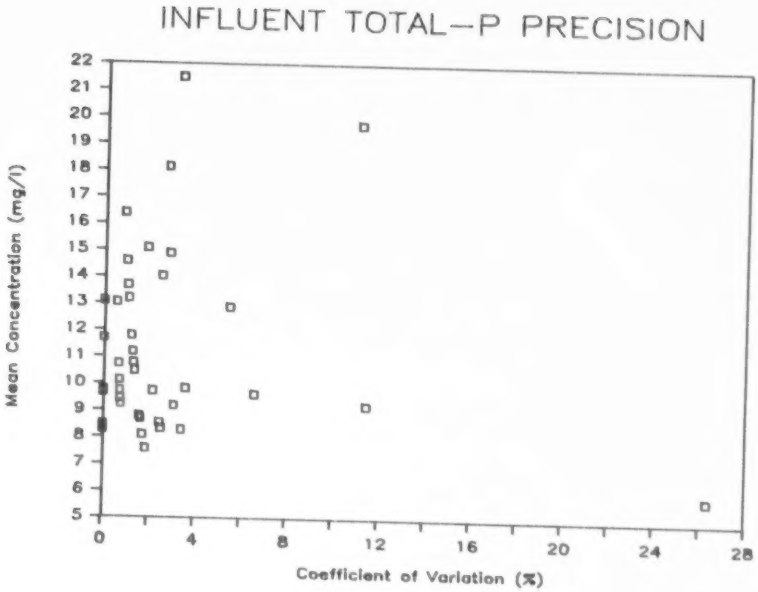
Bias

Spiked influent, effluent, and groundwater samples were analyzed and 98.4% (7.52), 102.8% (6.09), and 97.7% (8.9) recoveries were all deemed satisfactory compared with 100% (5) suggested in Table M-1 (numbers in parenthesis are standard deviations). The control standard recovery was 102% (2.8). The results are shown in Figure M-6.

TOTAL KJELDAHL NITROGEN

Precision

Figures M-7 shows the precision results for TKN. The CVs for all influent values were below the suggested 14% from Table M-1. The results for the effluent show the same difficulty experienced with the COD data discussed above. About half of the samples had a CV less than 14%, and the values ranged up to 120%.



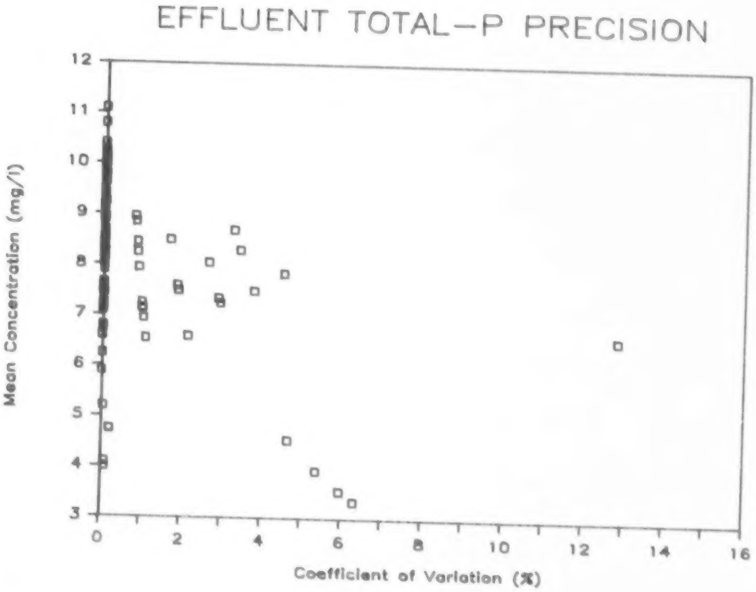


Figure M-5. Total phosphorus precision.

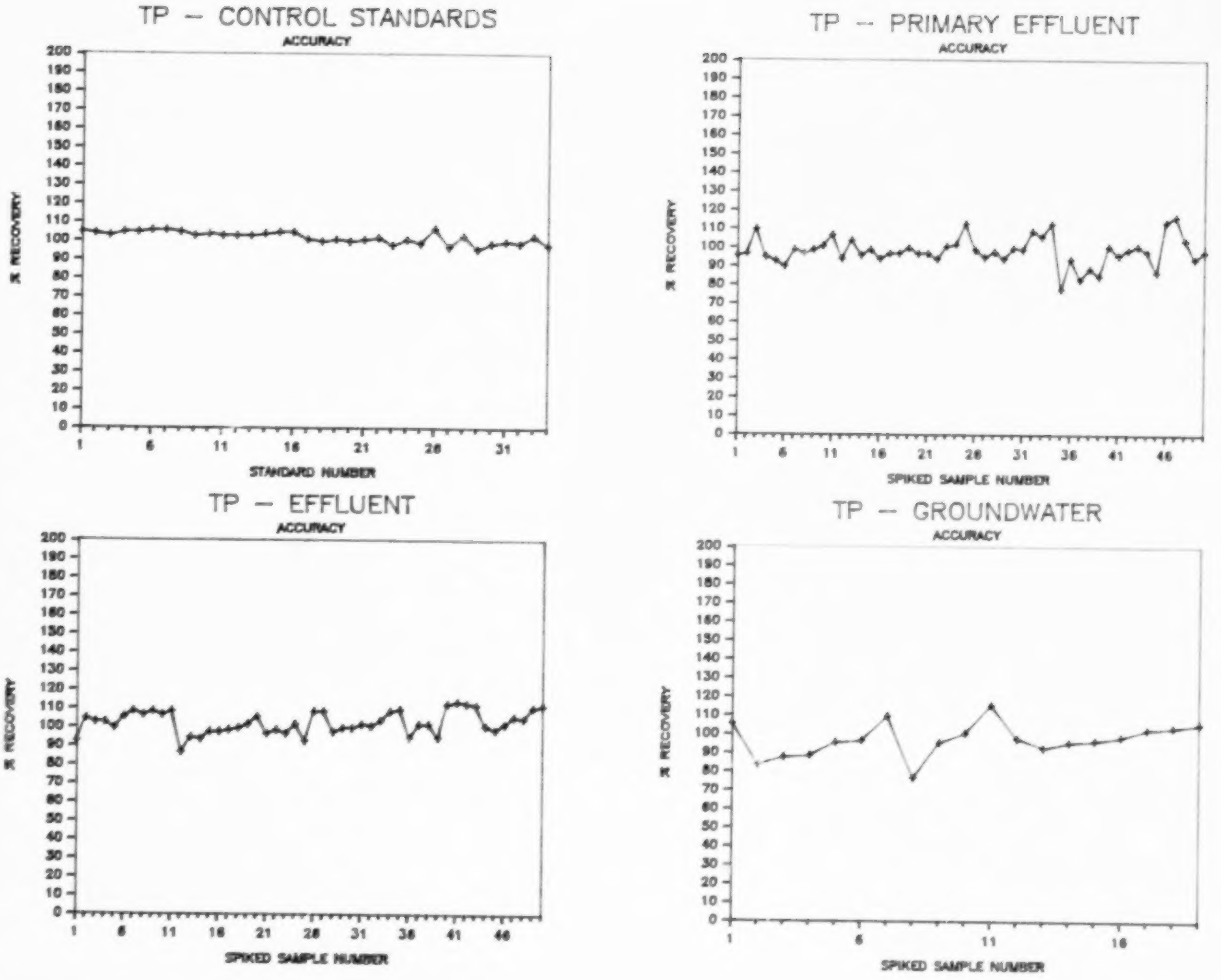
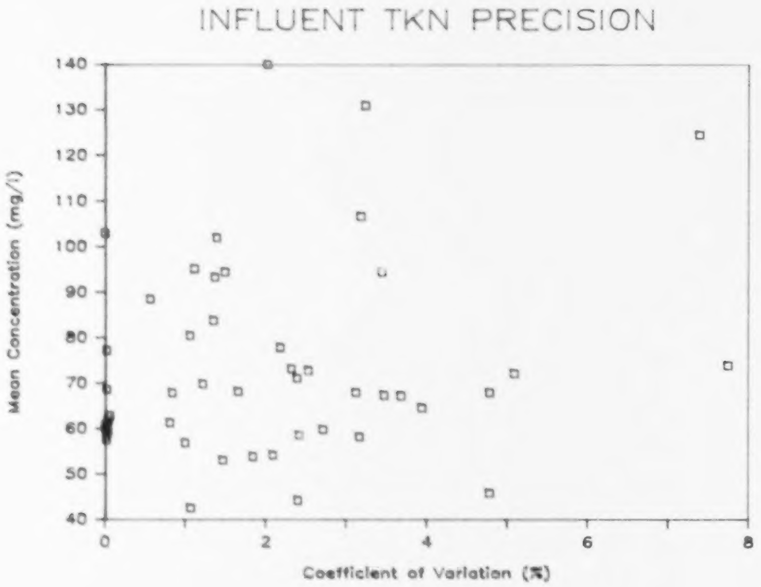
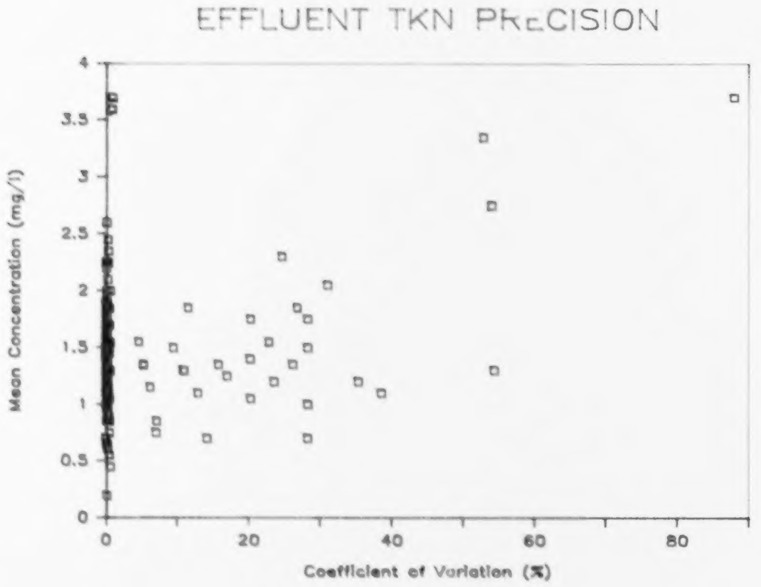


Figure M-6. Total phosphorus accuracy.



INFLUENT TKN PRECISION



EFFLUENT TKN PRECISION

Figure M-7. TKN precision.

Bias

Bias in the influent, effluent, and groundwater spike samples was 92.4% (10.6), 90.5% (9.78), and 93.8% (9.5), respectively. The control standard bias was 98.8% (3.0). Although none of the mean recoveries are different from 100%, the standard deviations of the spike samples are slightly high. Since the standard deviation of the control standard was within the acceptable range, these results may reflect the difficulty of recovering spikes from the wastewater matrix. The results are plotted in Figure M-8.

AMMONIA

Precision

Results for NH_3-N are similar to those for TKN. Individual results are plotted in Figure M-9. For the influent samples 97% of the duplicates' CV were below the suggested value of 14% in Table M-1, and all were below 16%. The effluent values were less precise with only about 30% of the samples having CV less than 14% and some values as high as 140%.

Bias

For ammonia, the spike recoveries for influent, effluent, and groundwater are 94.7% (5.21), 98.4% (5.11), and 93.5% (7.2), respectively and compare favorably with the values of 100% (5) given in Table M-1. The control standards' mean recovery was 98.1% (3.3). Figure M-10 shows the bias results for NH_3-N.

NITRATE + NITRITE

Precision

Precision results for NO_2 + NO_3-N are displayed in Figure M-11 for influent and effluent, respectively. For the effluent, the CVs were less than 14% for 99.2% of the duplicates, and all duplicate CVs were less than 20%. The influent values had CVs that were less than 25%.

Bias

Spike recoveries were measured for influent, effluent, and groundwater and found to be 91.8% (6.4), 89.8% (6.73), and 86.1% (7.5), respectively. All are slightly low. Since the control standard recovery was 103.2% (2.4), the slight bias in these results may show the difficulty in measuring oxidized nitrogen in a wastewater matrix. See Figure M-12 for these results.

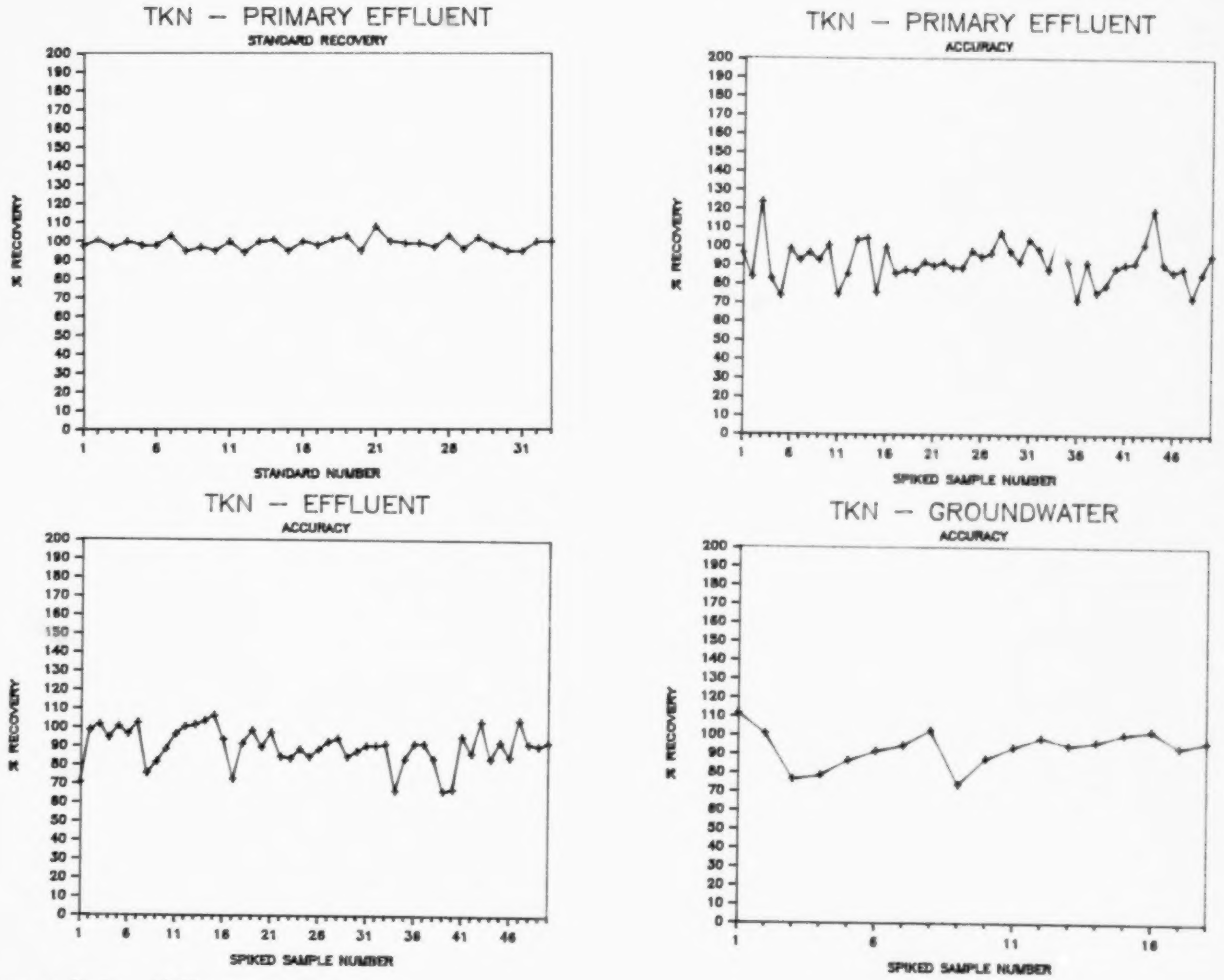


Figure M-3. TKN accuracy.

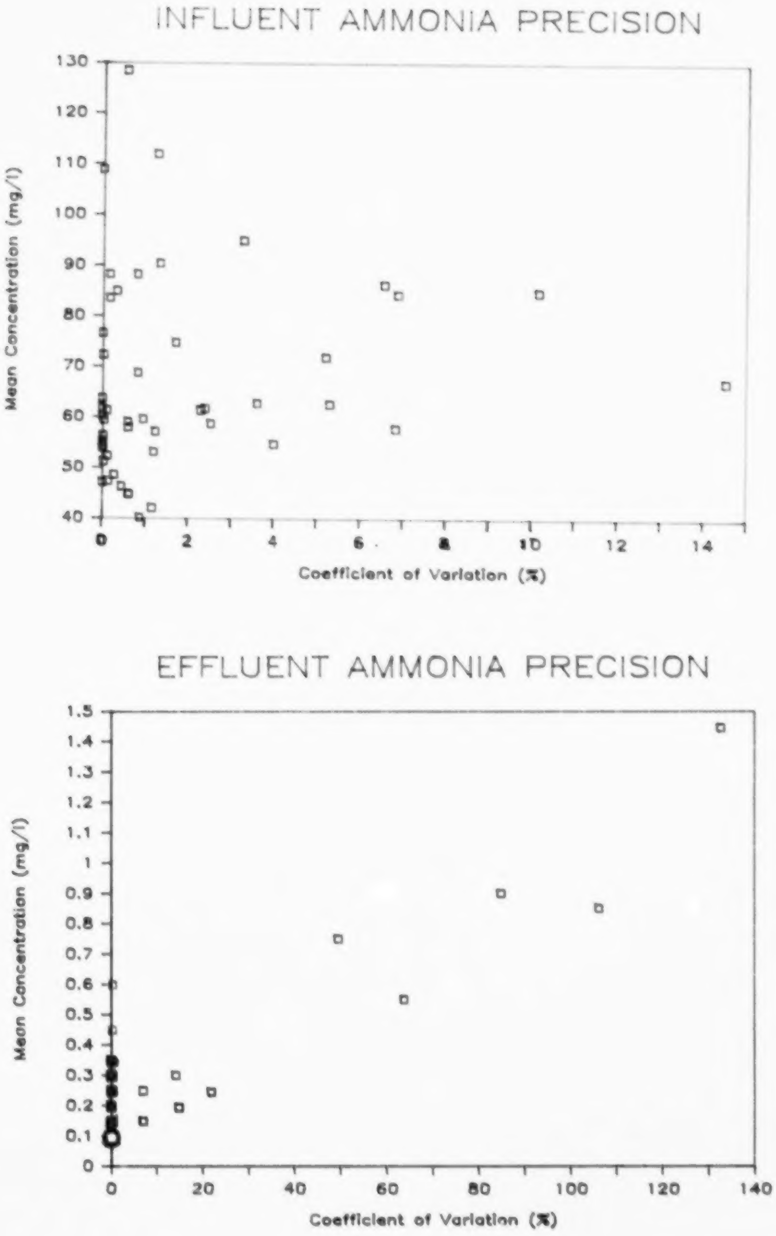


Figure M-9. Ammonia precision.

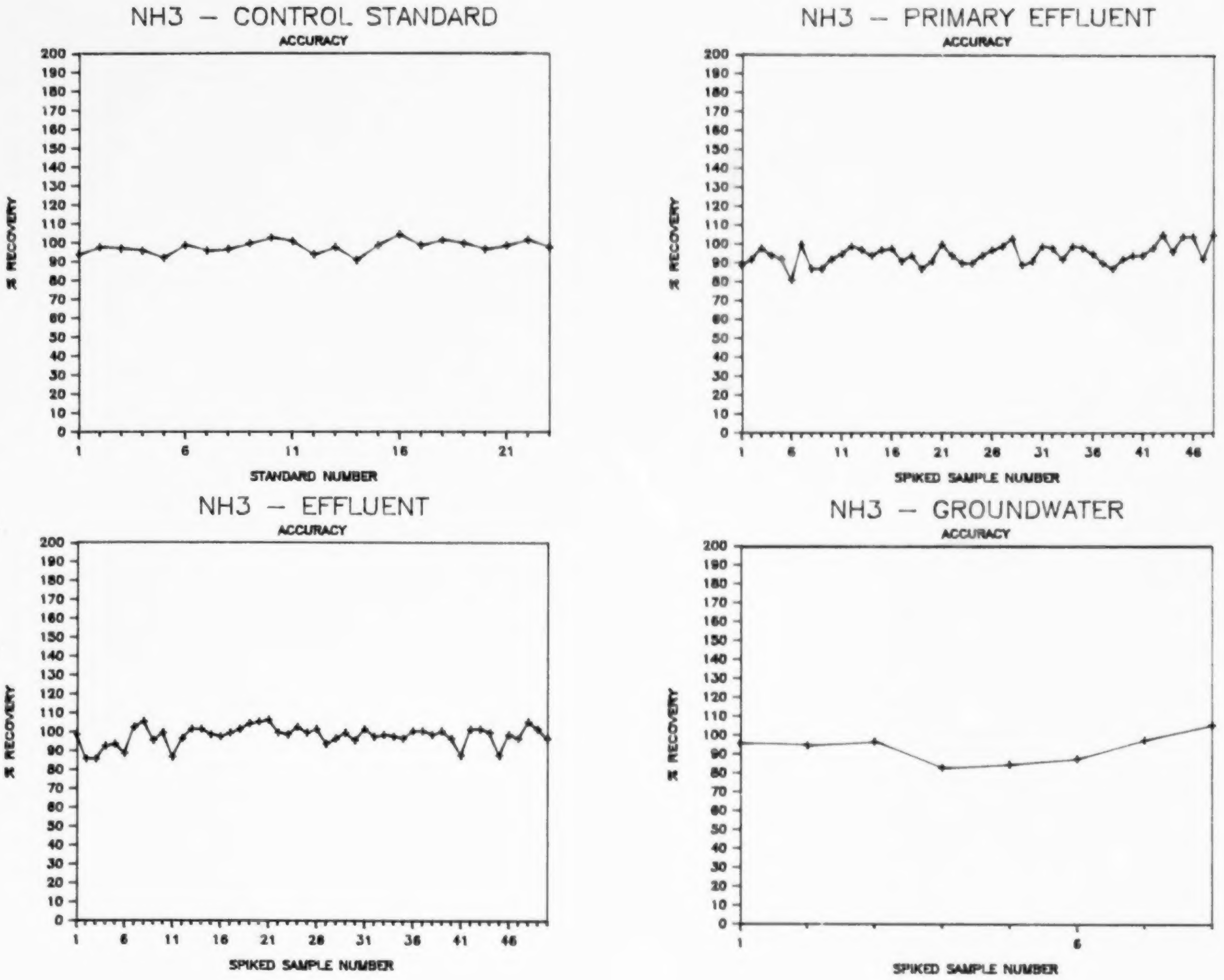
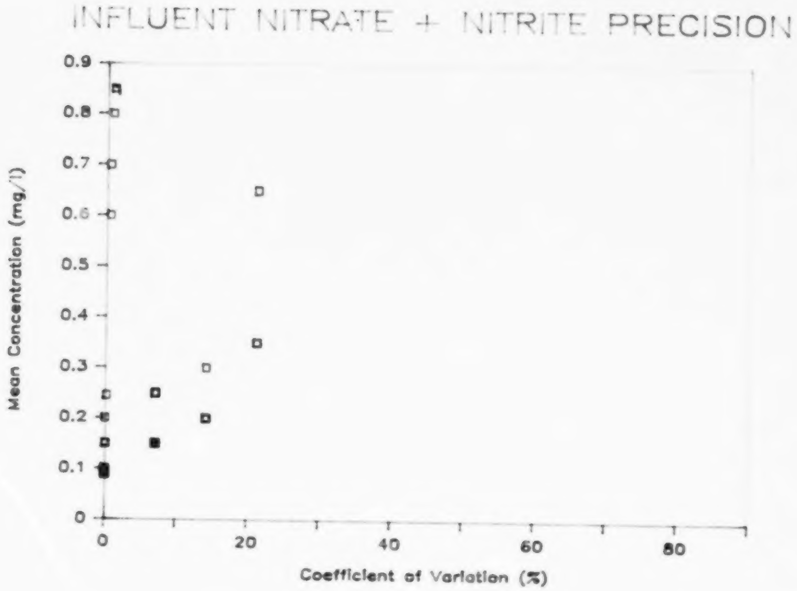


Figure M-10. Ammonia accuracy.



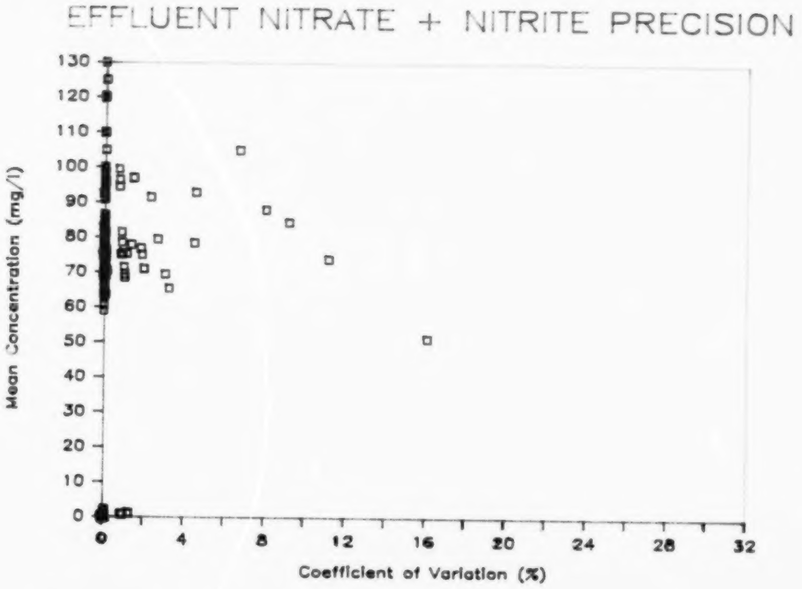


Figure M-11. $NO_2 + NO_3$-N precision.

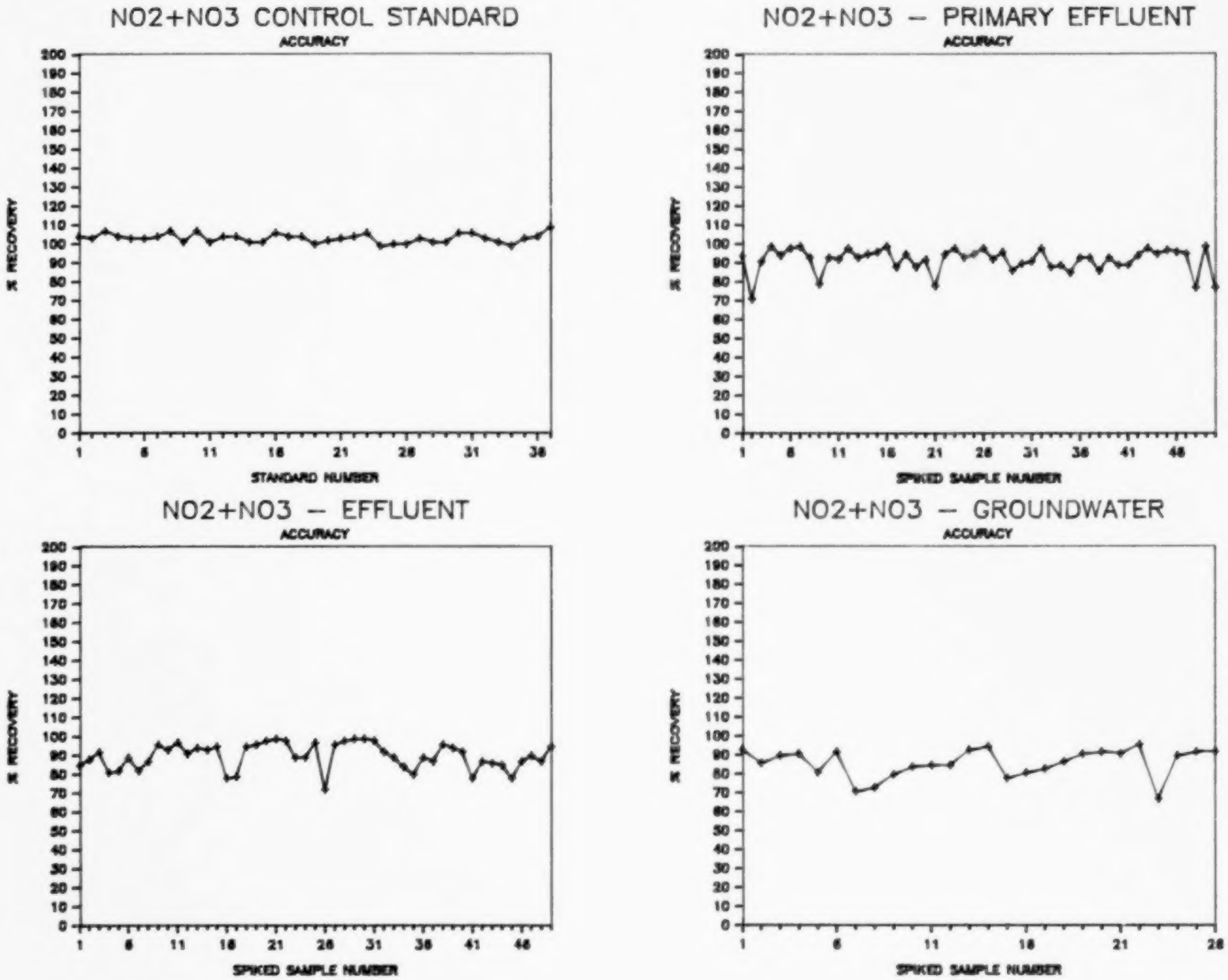


Figure M-12. NO_2 + NO_3 - N accuracy.

NITRITE

Precision

The precision results for NO_2-N are difficult to assess because most samples analyzed had no detectable amounts of NO_2-N. Only one sample out of 163 duplicates had detectable levels in both measurements. The sample had a CV of 9.2%, which compares favorably with the value of 14% given in Table M-1.

Bias

The spike recoveries for influent, effluent, and groundwater NO_2-N measurements were 99% (9.1), 92.8% (6.17), and 77.8% (10.6) showing that the recoveries of NO_2-N from groundwater are quite low. This suggests a problem with this method of nitrite analysis in groundwater. Because most groundwater NO_2-N concentrations were below the detection limit of 0.1 mg/l, this bias is felt to have little impact on the conclusions of this report. No control standard was analyzed for NO_2-N. These results are shown in Figure M-13.

FECAL COLIFORMS

Precision

Figure M-14 shows the precision results for fecal coliform measurements. The pooled CV for the duplicates is 5.4%, which compares with the suggested value of 26% in Table M-1. Ninety-eight percent of the duplicates' CVs were below 14%, while all values fell below 23%. The precision of these results is deemed satisfactory.

Bias

No spikes or standards were analyzed for fecal coliform.

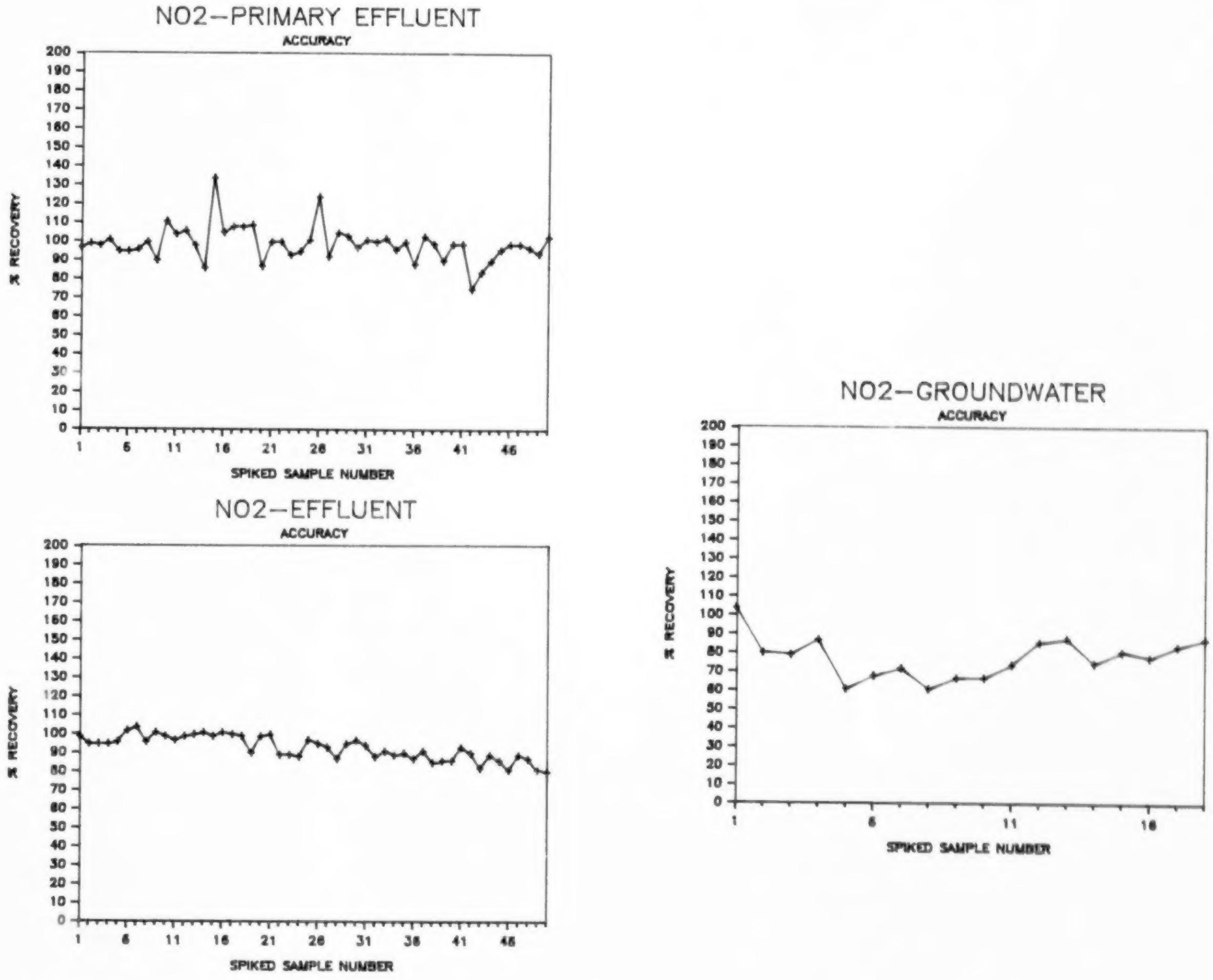
Completeness

An average of 96.2% of the fecal coliform samples yielded useful data. This compares well with the 90% target listed in Table M-1.

pH

Precision

The precision of the pH measurements was well within the requirement of ±0.1 pH units set out in Table M-1.


NO2-PRIMARY EFFLUENT

NO2-EFFLUENT

NO2-GROUNDWATER

Figure M-13. NO_2-N accuracy.

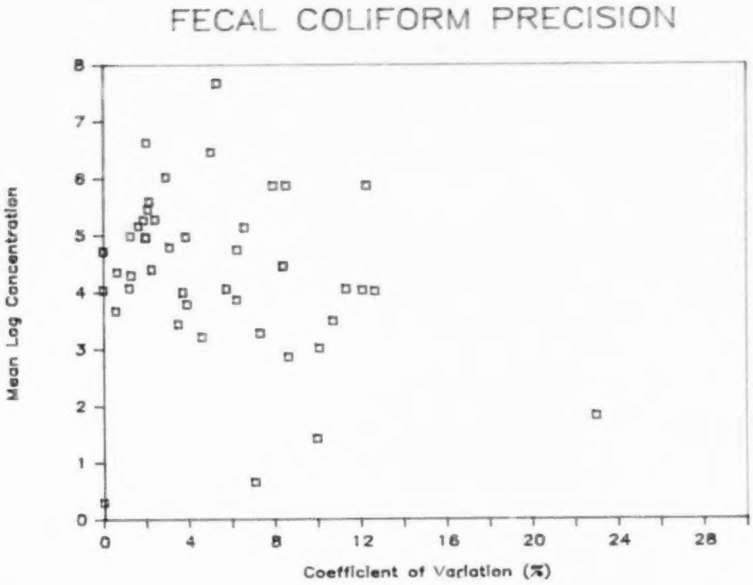


Figure M-14. Fecal coliform precision.

Bias

The results are felt to be unbiased because the pH meter was standardized using commercially prepared buffers before and after each set of measurements. Performance samples from EMSL were analyzed in the University of Cincinnati's lab and the results are tabulated in Table M-3.

TABLE M-3. PERFORMANCE EVALUATION SUMMARY OF pH

Date	Report value	True value	Acceptance limits	Warning limits	Performance evaluation
12/3/82	4.0	4.01	3.91-4.10	3.93-4.08	Accepted.
	9.0	9.19	8.92-9.38	8.98-9.32	Accepted.
5/27/83	7.1	7.10	6.95-7.26	6.99-7.23	Accepted.
	5.2	5.10	4.99-5.22	5.02-5.19	Warning.
12/14/83	6.90	6.87	6.74-7.01	6.78-6.98	Accepted.
	8.49	8.60	8.12-8.95	8.23-8.85	Accepted.
5/24/84	7.67	7.70	7.51-7.87	7.55-7.83	Accepted.
	4.84	4.80	4.68-4.91	4.71-4.88	Accepted.

Completeness

All desired measurements of pH are satisfactorily obtained.

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